GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL COMPLEX
KPC PROMENADE
NWC WEST RAMONA EXPRESSWAY
AND MAIN STREET
SAN JACINTO, CALIFORNIA

-Prepared By-

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March 11, 2016
(Revised 4/27/2016)

Project No. 644-15024
16-03-016

Latham Management Group
1600 East Florida Avenue, Suite 110
Hemet, California 92544

Subject: Geotechnical Investigation

Project: Proposed Commercial Complex
KPC Promenade
NWC West Ramona Expressway and Main Street
San Jacinto, California

Sladden Engineering is pleased to present the results of our geotechnical investigation performed for the commercial complex proposed for the subject site located on the northwest corner of Ramona Expressway and Main Street in the City of San Jacinto, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated January 12, 2016 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site in order to provide recommendations for foundation design and site preparation. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

The opinions, recommendations and design criteria presented in this report are based on our field exploration program, laboratory testing and engineering analyses. Based on the results of our investigation, it is our professional opinion that the proposed project should be feasible from a geotechnical perspective provided that the recommendations presented in this report are implemented into design and carried out through construction.

We appreciate the opportunity to provide service to you on this project. If you have any questions regarding this report, please contact the undersigned.

Respectfully submitted,
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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the commercial complex proposed for the subject site located on the northwest corner of Ramona Expressway and Main Street in the City of San Jacinto, California. The site is located at approximately 33.7856 degrees North latitude and 116.9389 degrees West longitude. The approximate location of the site is indicated on the Site Location Map (Figure 1).

Our investigation was conducted in order to evaluate the engineering properties of the subsurface materials, to evaluate their *in-situ* characteristics, and to provide engineering recommendations and design criteria for site preparation, foundation design and the design of various site improvements. This study also includes a review of published and unpublished geotechnical and geological literature regarding seismicity at and near the subject site.

PROJECT DESCRIPTION

Based on the provided Site Plan (Womer, 2016), it is our understanding that the proposed project will consist of constructing a total of eight (8) new commercial buildings on the subject site. The preliminary plans indicate the proposed commercial buildings include three (3) restaurant buildings, a hotel building, a gas station/ car wash, a fast food restaurant and (3) retail buildings. The project will also include paved parking areas, exterior concrete flatwork, underground utilities, landscape areas and various other improvements. For our analyses we expect that the proposed buildings will be of relatively lightweight wood-frame, reinforced masonry or steel-frame construction supported on conventional shallow spread footings and concrete slabs-on-grade.

Based on the relatively level nature of the site, it is apparent that grading will consist of minor cuts and fills in order to achieve the final pad elevations and to provide adequate gradients for site drainage. This does not include remedial grading consisting of the over-excavation and re-compaction of primary foundation bearing soil within the building areas. Upon completion of the precise grading plans, Sladden should be retained to review the plans in order to confirm that the recommendations presented within in this report are incorporated into the design of the proposed project.

Structural foundation loads were not available at the time of this report. Based on our experience with relatively lightweight structures, we expect that isolated column loads will be less than 30 kips and continuous wall loads will be less than 3.0 kips per linear foot. If these assumed loads vary significantly from the actual loads, we should be consulted to verify the applicability of the recommendations provided.

SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Exploration of the site was achieved, by drilling four (4) exploratory boreholes to depths between approximately 21 and 51 feet below the existing ground surface (bgs). Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- The excavation of four (4) exploratory boreholes to depths varying from approximately 21 to 51 feet bgs in order to characterize the subsurface soil conditions. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- The performance of laboratory testing on selected samples to evaluate their engineering characteristics.
- The review of geologic literature with respect to potential geologic hazards.
- The performance of engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

SITE CONDITIONS

The site is located on the northwest corner of Ramona Expressway and Main Street in the City of San Jacinto, California. The site consists of approximately 14.11 acres of undeveloped land. The site is currently vacant and covered in scattered weeds and grasses. The site is near the elevation of the adjacent properties and roadways and is bounded by vacant land to the north, Ramona Expressway to the east, Main Street to the south and Boxelder Way and Miracle Drive to the west.

The project site is relatively level with minimal surface gradients. According to the USGS 7.5′ San Jacinto Quadrangle map (USGS, 2015), the site is at an approximate elevation of 1590 feet above mean sea level (MSL).

No ponding water or surface seeps were observed at or near the site during our investigation conducted on January 29, 2016 Site drainage is controlled by sheet flow, surface infiltration and within City and/or County maintained storm drain systems located along nearby streets.

GEOLOGIC SETTING

The project site is located in the northern portion of the Peninsular Ranges Physiographic Province of California. The Peninsular Ranges are mountainous areas that extend from the western edge of the continental borderland to the Salton Trough and from the Transverse Ranges Physiographic Province in the north to the tip of Baja California in the south. The province is characterized by elongated, northwest-southeast trending mountain ranges and valleys and is truncated at its northern margin by the east-west grain of the Transverse Ranges.

The site has been mapped by Dibblee (2003) to be immediately underlain by alluvial deposits (Qa). The geologic setting for the site and site vicinity is presented on the Regional Geologic Map (Figure 2).

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling four (4) exploratory boreholes on the site. The approximate locations of the boreholes are illustrated on the Borehole Location Plan (Figure 3). The boreholes were advanced using a truck-mounted Mobile B-61 drill-rig equipped with 8-inch outside diameter hollow stem augers. A representative of Sladden was on-site to log the materials encountered and retrieve samples for laboratory testing and engineering analyses.

During our field investigation, artificial fill/disturbed soil and native alluvial materials were encountered to the maximum explored depth of approximately 51.5 feet bgs. Artificial fill soil was encountered near the surface within each of our bores. The artificial fill soil was generally less than two (2) feet in depth within the areas of our bores. The native soil consists primarily of gravelly sand (SP) and silty sand (SM). Sampler penetration resistance as measured by field blow counts indicates that density generally increases with depth.

Groundwater was not encountered within our boreholes. Based upon our bores and our review of CDWR (2016), it is our opinion that groundwater is at a sufficient depth as not to be a factor during construction of the proposed structures.

The final logs represent our interpretation of the contents of the field logs, and the results of the laboratory observations and tests of the field samples. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types although the transitions may be gradual and/or variable across the site.

SEISMICITY AND FAULTING

The southwestern United States is a tectonically active and structurally complex region, dominated by northwest trending dextral faults. The faults of the region are often part of complex fault systems, composed of numerous subparallel faults that splay or step from the main fault traces. Strong seismic shaking could be produced by any of these faults during the design life of the proposed project.

We consider the most significant geologic hazard to the project to be the potential for moderate to strong seismic shaking that is likely to occur during the design life of the project. The proposed project is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene epoch (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

Table 1 lists the closest known active faults that were generated in part using the EQFAULT computer programs (Blake, 2000), as modified using the fault parameters from The Revised 2002 California Probabilistic Seismic Hazard Maps (Cao et al, 2003). This table does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any of the other faults in the region.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
San Jacinto- San Jacinto Valley	2.1	6.9
San Jacinto-Anza	5.5	7.2
San Andreas-Southern	25.1	7.2
San Andreas-San Bernardino	25.1	7.5
Pinto Mountain	36.3	7.2
Elsinore – Temecula	37.6	6.8
San Jacinto-San Bernardino	37.7	6.7
Elsinore - Glen Ivy	41.0	6.8
San Andreas- Coachella	45.9	7.2
Elsinore - Julian	46.0	7.1

2013 CBC SEISMIC DESIGN PARAMETERS

Sladden has reviewed the 2013 California Building Code (CBC) and summarized the current seismic design parameters for the proposed structures. The seismic design category for a structure may be determined in accordance with Section 1613 of the 2013 CBC or ASCE7. According to the 2013 CBC, Site Class D may be used to estimate design seismic loading for the proposed structures. The 2013 CBC Seismic Design Parameters are summarized below. The project Design Map Reports are included within Appendix C.

Risk Category (Table 1.5-1): I/II/III
Site Class (Table 1613.3.2): D
S_s (Figure 1613.3.1): 2.429g
S_1 (Figure 1613.3.1): 1.071g
F_a (Table 1613.3.3(1)): 1.0
F_v (Table 1613.5.3(2)): 1.5
S_{ms} (Equation 16-37 {F_a X S_s}): 2.429g
S_{m1} (Equation 16-38 {F_v X S_1}): 1.606g
S_{DS} (Equation 16-39 {2/3 X S_{ms}}): 1.620g
S_{D1} (Equation 16-40 {2/3 X S_{m1}}): 1.071g
Seismic Design Category: E

GEOLOGIC HAZARDS

The subject site is located in an active seismic zone and will likely experience strong seismic shaking during the design life of the proposed project. In general, the intensity of ground shaking will depend on several factors including: the distance to the earthquake focus, the earthquake magnitude, the response characteristics of the underlying materials, and the quality and type of construction. Geologic hazards and their relationship to the site are discussed below.

I. Surface Rupture. Surface rupture is expected to occur along preexisting, known active fault traces. However, surface rupture could potentially splay or step from known active faults or rupture along unidentified traces. Based on review of Jennings (1994), CDOC (2016) and Rodgers (1965) faults are not mapped on the site. In addition, no signs of active surface faulting were observed during our review of non-stereo digitized photographs of the site and site vicinity (Google, 2016; Terra Server 2002). Finally, no signs of active surface rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified on-site during our field investigation. Therefore, it is our opinion that risks associated with primary surface ground rupture should be considered "low".

II. Ground Shaking. The site has been subjected to past ground shaking by faults that traverse through the region. Strong seismic shaking from nearby active faults is expected to produce strong seismic shaking during the design life of the proposed project. A probabilistic approach was employed to the estimate the peak ground acceleration (a_{max}) that could be experienced at the site. Based on the USGS Interactive Deaggregation (USGS, 2008) and shear wave velocity (Vs30) of 360 m/s, the site could be subjected to ground motions on the order of 0.63g (USGS, 2015a, 2015b). The peak ground acceleration at the site is judged to have a 475 year return period and a 10 percent chance of exceedence in 50 years.

III. Liquefaction/Seismic Settlement. Liquefaction is the process in which loose, saturated granular soil loses strength as a result of cyclic loading. The strength loss is a result of a decrease in granular sand volume and a positive increase in pore pressures. Generally, liquefaction can occur if all of the following conditions apply: liquefaction-susceptible soil, groundwater within a depth of 50 feet or less, and strong seismic shaking.

According to the County of Riverside, the site is situated within a "Moderate" liquefaction potential zone (RCPR, 2016). Based on the depth to groundwater (CDWR, 2016) Sladden anticipates hazards resulting from liquefaction to be "negligible".

IV. Tsunamis and Seiches. Because the site is situated at an elevated inland location, and is not immediately adjacent to any impounded bodies of water, risk associated with tsunamis and seiches is considered negligible.

V. Slope Failure, Landsliding, Rock Falls. The site is situated on relatively level ground and is not immediately adjacent to any slopes or hillsides that could be potentially susceptible to slope instability. No signs of slope instability in the form of landslides, rock falls, earthflows or slumps were observed at or near the subject site during our investigation. As such, risks associated with slope instability should be considered "negligible".

VI. Expansive Soil. Expansion Index testing of select samples was performed in order to evaluate the expansion potential of the materials underlying the site. Based the results of our laboratory testing (EI=8), the materials present near the ground surface are considered to have a "very low" expansion potential. Accordingly, risk of structural damage caused by volumetric changes in the subgrade soil is considered "low". However, the surface soil should be tested subsequent to grading and final foundation and slab design should be based upon post-grading expansion test results.

VII. Settlement. Settlement resulting from the anticipated foundation loads should be tolerable provided that the recommendations included in this report are considered in foundation design and construction. The estimated ultimate settlement is calculated to be approximately one inch when using the recommended bearing values. As a practical matter, differential settlement between footings can be assumed as one-half of the total settlement.

VIII. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation. Risks associated with flooding and erosion should be considered evaluated and mitigated by the project design Civil Engineer.

CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project should be feasible from a geotechnical perspective provided that the recommendations provided in this report are incorporated into design and carried out through construction. The main geotechnical concern in the design and construction of the proposed project is the presence of artificial fill soil.

Because of the presence of undocumented artificial fill soil and the somewhat soft and compressible condition of some of the near surface soil, remedial grading including overexcavation and recompaction is recommended for the proposed building and foundation areas. We recommend that remedial grading within the proposed building areas include over-excavation and/or re-compaction of the artificial fill soil and the primary foundation bearing soil. Specific recommendations for site preparation are presented in the Earthwork and Grading section of this report.

Groundwater was not encountered within our bores to a depth of 51 feet. Therefore, it is our opinion that groundwater should not be a factor during the construction of the proposed project.

Caving did occur to varying degrees within each of our exploratory bores and the surface soil may be susceptible to caving within deeper excavations. All excavations should be constructed in accordance with the normal CalOSHA excavation criteria. On the basis of our observations of the materials encountered, we anticipate that the subsoil will conform to that described by CalOSHA as Type B or C. Soil conditions should be verified in the field by a "Competent person" employed by the Contractor.

The following recommendations present more detailed design criteria that have been developed on the basis of our field and laboratory investigation.

EARTHWORK AND GRADING

All earthwork including excavation, backfill and preparation of the surface soil, should be performed in accordance with the geotechnical recommendations presented in this report and portions of the local regulatory requirements, as applicable. All earth work should be performed under the observation and testing of a qualified soil engineer. The following geotechnical engineering recommendations for the proposed project are based on observations from the field investigation program, laboratory testing and geotechnical engineering analyses.

a. Stripping. Areas to be graded should be cleared of the scattered weeds and surface vegetation. All areas scheduled to receive fill should be cleared of surface improvements, artificial fill and any unsuitable matter. The unsuitable materials should be removed from the site. Existing artificial fill soil should be removed in its entirety and replaced as engineered fill. Voids left by obstructions should be properly backfilled in accordance with the compaction recommendations of this report.

b. Preparation of Building Areas. In order to achieve firm and uniform bearing conditions, we recommend over-excavation and re-compaction throughout the building areas. All artificial fill and native low density near surface soil should be removed to competent native soil expected at depths of approximately 4 to 5 feet below the existing ground surface or to a minimum depth of 4 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally, a minimum of five feet beyond perimeter wall foundations. The exposed surface soil should then be scarified, moisture conditioned to within two percent of optimum moisture content, and compacted to at least 90 percent relative compaction. The previously removed soil may then be replaced as described below.

c. Compaction. Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain irreducible matter greater than three inches in maximum dimension. All fill materials should be placed in thin lifts, not exceeding six inches in a loose condition. If import fill is required, the material should be of a non-expansive nature and should meet the following criteria:

Plastic Index	Less than 12
Liquid Limit	Less than 35
Percent Soil Passing #200 Sieve	Between 15% and 35%
Maximum Aggregate Size	3 inches

The subgrade and all fills material should be compacted with acceptable compaction equipment, to at least 90 percent relative compaction. The bottom of the exposed subgrade should be observed by a representative of Sladden Engineering prior to fill placement. Compaction testing should be performed on all lifts in order to ensure proper placement of the fill materials. Table 2 provides a summary of the excavation and compaction recommendations.

TABLE 2
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Excavation and recompaction within the building envelope and extending laterally at least 5 feet beyond the building limits and to competent native soil or a minimum depth of 4 feet below the bottom of the footings, whichever is deeper.
Native / Import Engineered Fill	Place in thin lifts not exceeding 6 inches in a loose condition at optimum moisture content and compact to a minimum of 90 percent relative compaction.
Asphalt Concrete Sections	Compact the top 12 inches to at least 95 percent compaction within 2 percent of optimum moisture content.

*Actual depth may vary and should be determined by a representative of Sladden Engineering in the field during construction.

d. Shrinkage and Subsidence. Volumetric shrinkage of the material that is excavated and replaced as controlled compacted fill should be anticipated. We estimate that this shrinkage could vary from 15 to 20 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 and 2 tenths of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

FOUNDATIONS: CONVENTIONAL SHALLOW SPREAD FOOTINGS

Exterior footings should extend at least 12 inches beneath lowest adjacent grade and interior footings should extend at least 12 inches below slab subgrade. Isolated square or rectangular footings at least 2 feet square and continuous footings at least 12 inches wide may be designed using allowable bearing pressures of 2000 and 1800 pounds per square foot, respectively. The allowable bearing pressure may be increased by approximately 250 psf for each additional 1 foot of width and 250 psf for each additional 6 inches of depth, if desired. The maximum allowable bearing pressure should be limited to 3000 psf unless confirmed by Sladden Engineering subsequent to performing specific settlement calculations. The allowable bearing pressures are for dead and frequently applied live loads and may be increased by 1/3 to resist wind, seismic or other transient loading.

The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces. All footings should be reinforced in accordance with the project structural engineer's recommendations.

Based on the allowable bearing pressures recommended above, total settlement of the shallow footings are anticipated to be less than one inch, provided that foundation preparation conforms to the recommendations provided in this report. Differential settlement is anticipated to be approximately one-half the total settlement for similarly loaded footings spaced approximately 40 feet apart.

Resistance to lateral loads may be provided by a combination of friction acting at the base of the slabs or foundations and passive earth pressure along the sides of the foundations. A coefficient of friction of 0.40 between soil and concrete may be used for dead load forces only. A passive earth pressure of 250 pounds per square foot, per foot of depth, may be used for the sides of footings that are placed against properly compacted native soil. Passive earth pressure should be ignored within the upper 1 foot except where confined.

All footing excavations should be observed by a representative of the project geotechnical consultant to verify adequate embedment depths prior to placement of forms, steel reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, disturbed, sloughed or moisture-softened soil and/or any construction debris should be removed prior to concrete placement. Excavated soil generated from footing and/or utility trenches should not be stockpiled within the building envelope or in areas of exterior concrete flatwork.

SLABS-ON-GRADE

In order to reduce the risk of heave, cracking and settlement, concrete slabs-on-grade must be placed on properly compacted fill as outlined in the previous sections of this report. The slab subgrades should remain near optimum moisture content and should not be permitted to dry, prior to concrete placement. All slab subgrades should be firm and unyielding. Disturbed soils should be removed and then replaced and compacted to a minimum of 90 percent relative compaction.

Slab thickness and reinforcement should be determined by the structural engineer. All slab reinforcement should be supported on concrete chairs to ensure that reinforcement is placed at slab mid-height. Considering the expected uses, we recommend a minimum slab thickness of 4.0 inches.

Slabs with moisture sensitive surfaces should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 10-mil Visqueen. All laps within the membrane should be sealed and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete and to limit damage. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface can not be achieved by grading, consideration should be given to placing a 1-inch thick leveling course of sand across the pad surface prior to placement of the membrane.

PRELIMINARY PAVEMENT DESIGN

Asphalt concrete pavements should be designed in accordance with Topic 610 of the Caltrans Highway Design Manual based on R-Value and Traffic Index. The R-Value of the near surface soil is expected to exceed 40. On-site soil and any imported soil should be tested for R-Value prior to establishing final pavement design sections.

For preliminary pavement design, Traffic Indices (TI) of 5.0 and 6.5 were used for the light duty and heavy duty pavements, respectively. We assumed Asphalt Concrete (AC) over Class II Aggregate Base (AB). The preliminary flexible pavement layer thickness is as follows:

RECOMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS

Pavement Material	Recommended Thickness	
	TI=5.0	TI=6.5
Asphalt Concrete Surface Course	3.0 inches	4.0 inches
Class II Aggregate Base Course	4.0 inches	6.0 inches
Compacted Subgrade Soil	12 inches	12 inches

Asphalt concrete should conform to Sections 203 and 302 of the latest edition of the Standard Specifications for Public Works Construction (Caltrans or Greenbook). Class II aggregate base should conform to Section 26 of the Caltrans Standard Specifications or Greenbook, latest edition. The aggregate base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Method D 1557.

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 20 parts per million (ppm). The soil is considered to have a "negligible" corrosive potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes should not be necessary. Soluble sulfate content of the surface soil should be reevaluated after grading and appropriate concrete mix designs should be established based upon post-grading test results.

The pH level of the surface soil was determined to be 8.2. Based on soluble chloride concentration testing (30 ppm), the soil is considered to have a "negligible" corrosive potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 3400 ohm-cm that suggests the site soil is considered to have a "moderate" corrosive potential with respect to ferrous metal installations. A corrosion expert should be consulted regarding appropriate corrosion protection measures.

UTILITY TRENCH BACKFILL

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Trench backfill materials should be placed in lifts no greater than six inches in a loose condition, moisture conditioned (or air-dried) as necessary to achieve near optimum moisture content, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should test the backfills to verify adequate compaction.

EXTERIOR CONCRETE FLATWORK

To minimize cracking of concrete flatwork, the subgrade soil below concrete flatwork areas should first be compacted to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soil.

DRAINAGE

All final grades should be provided with positive gradients away from foundations to provide rapid removal of surface water runoff to an adequate discharge point. No water should be allowed to be pond on or immediately adjacent to foundation elements. In order to reduce water infiltration into the subgrade soil, surface water should be directed away from building foundations to an adequate discharge point. Subgrade drainage should be evaluated upon completion of the precise grading plans and in the field during grading.

LIMITATIONS

The findings and recommendations presented in this report are based upon an interpolation of the soil conditions between the exploratory boring locations and extrapolation of these conditions throughout the proposed building area. Should conditions encountered during grading appear different than those indicated in this report, this office should be notified.

The use of this report by other parties or for other projects is not authorized. The recommendations of this report are contingent upon monitoring of the grading operation by a representative of Sladden Engineering. All recommendations are considered to be tentative pending our review of the grading operation and additional testing, if indicated. If others are employed to perform any soil testing, this office should be notified prior to such testing in order to coordinate any required site visits by our representative and to assure indemnification of Sladden Engineering.

We recommend that a pre-job conference be held on the site prior to the initiation of site grading. The purpose of this meeting will be to assure a complete understanding of the recommendations presented in this report as they apply to the actual grading performed.

ADDITIONAL SERVICES

Once completed, final project plans and specifications should be reviewed by use prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the Soil Engineer during construction to document that foundation elements are founded on/or penetrate into the recommended soil, and that suitable backfill soil is placed upon competent materials and properly compacted at the recommended moisture content.

Tests and observations should be performed during grading by the Soil Engineer or his representative in order to verify that the grading is being performed in accordance with the project specifications. Field density testing shall be performed in accordance with acceptable ASTM test methods. The minimum acceptable degree of compaction should be 90 percent for subgrade soils and 95 percent for Class II aggregate base as obtained by the ASTM D1557 test method. Where testing indicates insufficient density, additional compactive effort shall be applied until retesting indicates satisfactory compaction.

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FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PLAN

Sladden Engineering	SITE LOCATION MAP		FIGURE
	Project Number:	644-15024	1
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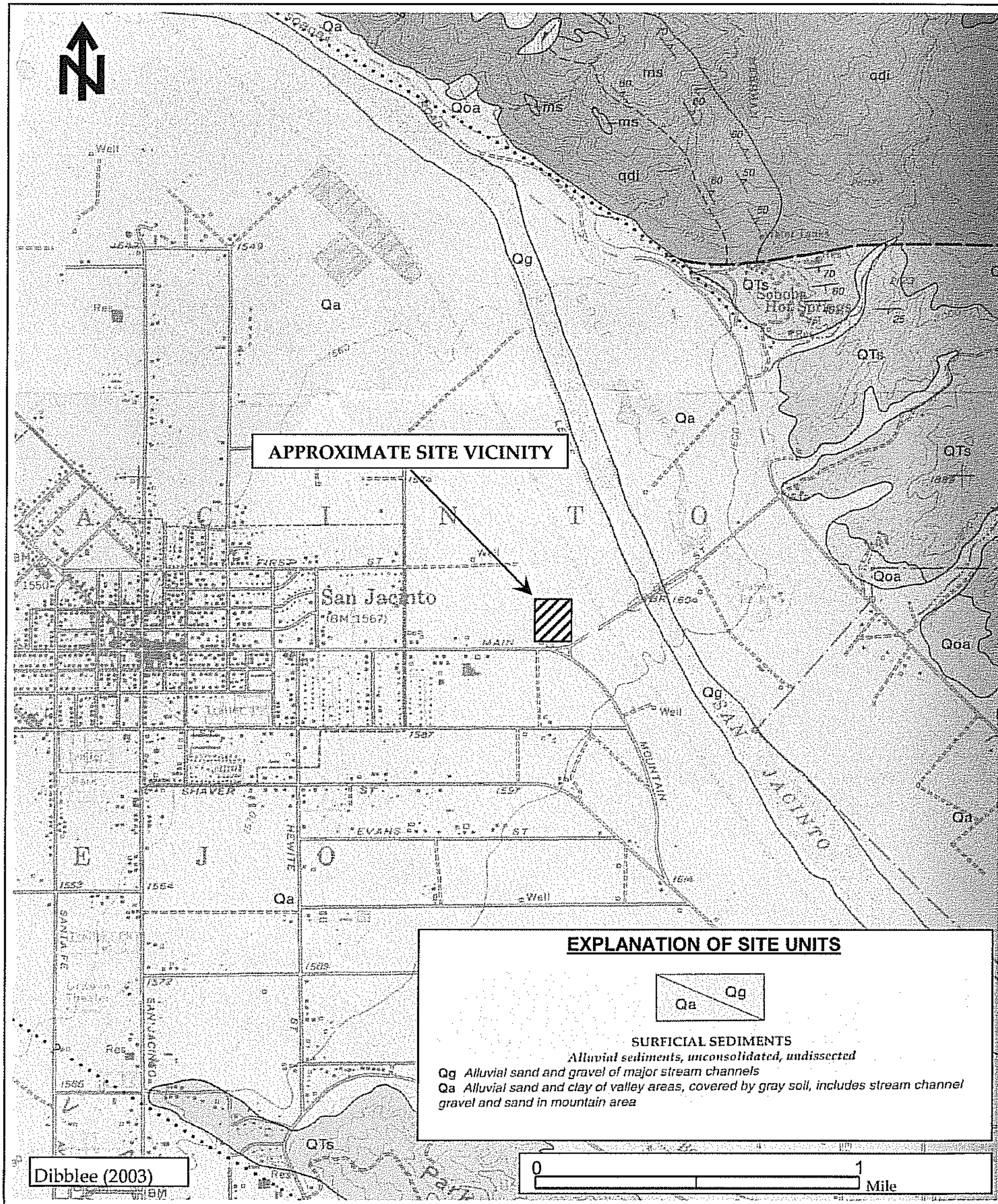

Sladden Engineering	REGIONAL GEOLOGIC MAP		FIGURE 2
	Project Number:	644-15024	
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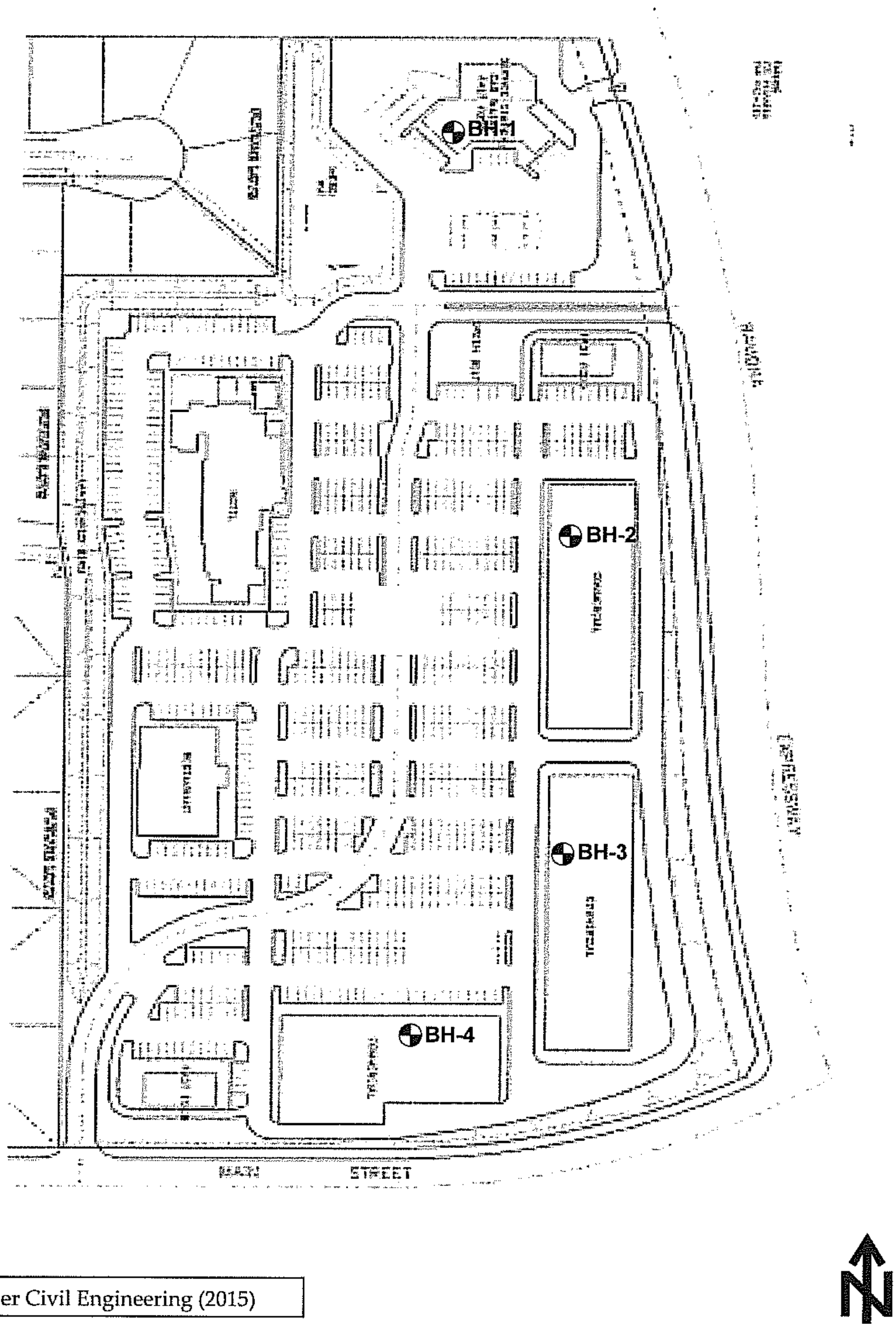

Blaine A. Womer Civil Engineering (2015)

Sladden Engineering

BOREHOLE LOCATION PLAN

Project Number:	644-15024
Report Number:	16-03-016
Date:	March 11, 2016

FIGURE

3

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation four (4) exploratory bores were excavated on January 29, 2016 utilizing a truck mounted hollow stem auger rig (Mobile B-61). Continuous logs of the materials encountered were made by a representative of Sladden Engineering. Materials encountered in the boreholes were classified in accordance with the Unified Soil Classification System which is presented in this appendix.

Representative undisturbed samples were obtained within our bores by driving a thin-walled steel penetration sampler (California split spoon sampler) or a Standard Penetration Test (SPT) sampler with a 140 pound automatic-trip hammer dropping approximately 30 inches (ASTM D1586). The number of blows required to drive the samplers 18 inches was recorded in 6-inch increments and blowcounts are indicated on the boring logs.

The California samplers are 3.0 inches in diameter, carrying brass sample rings having inner diameters of 2.5 inches. The standard penetration samplers are 2.0 inches in diameter with an inner diameter of 1.5 inches. Undisturbed samples were removed from the sampler and placed in moisture sealed containers in order to preserve the natural soil moisture content. Bulk samples were obtained from the excavation spoils and samples were then transported to our laboratory for further observations and testing.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN No.200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN No.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY-GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN No.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN No.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, CLEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS: LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

EXPLANATION OF BORE LOG SYMBOLS

California Split-spoon Sample

Unrecovered Sample

Standard Penetration Test Sample

Groundwater depth

Note: The stratification lines on the borelogs represent the approximate boundaries between the soil types; the transitions may be gradual.

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	1/29/2016
Elevation:	1590 Ft (MSL)	Boring No:	BH-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
									Silty Sand (SM); grayish brown, slightly moist, fine-to coarse-grained (Fill).
	3/5/5	1	8	4.8	1.0	110.9	2		Gravelly Sand (SM); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	4/4/4			2.1	1.2	104.6	6		Gravelly Sand (SP); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	4/5/5			9.4	5.0		10		Gravelly Sand (SM); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	8/10/10			3.1	2.7	106.8	16		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	8/10/10			3.9	2.9		20		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	11/14/16			3.6	3.4	117.2	26		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	9/12/11			2.6	2.9		30		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	13/20/23			4.2	2.9	106.3	36		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	8/12/12			15.5	9.2		40		Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-to coarse-grained with gravel, trace of clay (Qa).
	18/21/25			3.1	1.5	115.3	46		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	12/13/13			16.8	6.8		50		Silty Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained with gravel (Qa).

Completion Notes:
Terminated at ~ 51.5 feet bgs.
No Bedrock Encountered.
No Groundwater Encountered.

PROPOSED COMMERCIAL COMPLEX
NWC WEST RAMONA EXPRESSWAY AND MAIN STREET

Project No: 644-15024
Report No: 16-03-016

SLADDEN ENGINEERING	BORE LOG			
	Drill Rig:	Mobil B-61	Date Drilled:	1/29/2016
	Elevation:	1591 Ft (MSL)	Boring No:	BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–2		Silty Sand (SM); grayish brown, slightly moist, fine-to coarse-grained with gravel (Fill).
	1/2/1			2.0	1.9		5		Gravelly Sand (SP); grayish brown, slightly moist, very loose, fine-to coarse-grained (Qa).
	6/9/8			1.4	1.4	108.1	10		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	5/7/8			3.1	3.7		15		Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
	8/10/10			2.8	4.2	106.8	20		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
									Terminated at ~21.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

Completion Notes:

PROPOSED COMMERCIAL COMPLEX
NWC WEST RAMONA EXPRESSWAY AND MAIN STREET

Project No: 644-15024
Report No: 16-03-016

SLADDEN ENGINEERING	BORE LOG			
	Drill Rig:	Mobil B-61	Date Drilled:	1/29/2016
	Elevation:	1593 Ft (MSL)	Boring No:	BH-3

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–2		Gravelly Sand (SP); grayish brown, slightly moist, fine to coarse grained (Fill).
							2–4		Gravelly Sand (SP); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	1/3/4			5.8	2.8		5		Gravelly Sand (SP); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	6/8/8			2.0	2.5	102.6	10		Gravelly Sand (SP); grayish brown, slightly moist, loose, fine-to coarse-grained (Qa).
	5/7/7			2.3	2.8		15		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	10/13/13			3.1	2.5	109.9	20		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	9/12/13			3.6	2.8		25		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	15/15/15			2.6	1.6	113.4	30		Gravelly Sand (SP); grayish brown, slightly moist, medium dense, fine-to coarse-grained (Qa).
	1/4/5			53.5	24.4		35		Sandy Silt (ML); grayish brown, moist, stiff, low plasticity with gravel and trace clay (Qa).
	13/11/16			11.8	6.3	110.2	40		Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
	4/9/12			59.0	21.4		45		Sandy Clay (CL); dark grayish brown, moist, very stiff, medium plasticity with gravel and trace silt (Qa).
	14/22/22			29.1	22.6	106.6	50		Silty Sand (SM); dark grayish brown, wet, medium dense with gravel (Qa).

Completion Notes:
Terminated at ~ 51.5 feet bgs.
No Bedrock Encountered.
No Groundwater Encountered.

PROPOSED COMMERCIAL COMPLEX
NWC WEST RAMONA EXPRESSWAY AND MAIN STREET

Project No: 644-15024
Report No: 16-03-016

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	1/29/2016
Elevation:	1596 Ft (MSL)	Boring No:	BH-4

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
									Silty Sand (SM); grayish brown, slightly moist, fine-to coarse-grained with gravel (Fill).
	6/8/11			3.0	1.6	110.3			Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
	5/5/5			2.0	3.3				Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
	8/11/12			3.3	3.6	102.6			Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
	9/10/11			2.3	2.6				Gravelly Sand (SP); gray, slightly moist, medium dense, fine-to coarse-grained (Qa).
									Terminated at ~21.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Depth scale (Feet): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

Completion Notes:

PROPOSED COMMERCIAL COMPLEX
NWC WEST RAMONA EXPRESSWAY AND MAIN STREET

Project No: 644-15024
Report No: 16-03-016

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Representative bulk and relatively undisturbed soil samples were obtained in the field and returned to our laboratory for additional observations and testing. Laboratory testing was generally performed in two phases. The first phase consisted of testing in order to determine the compaction of the existing natural soil and the general engineering classifications of the soils underlying the site. This testing was performed in order to estimate the engineering characteristics of the soil and to serve as a basis for selecting samples for the second phase of testing. The second phase consisted of soil mechanics testing. This testing including consolidation, shear strength and expansion testing was performed in order to provide a means of developing specific design recommendations based on the mechanical properties of the soil.

CLASSIFICATION AND COMPACTION TESTING

Unit Weight and Moisture Content Determinations: Each undisturbed sample was weighed and measured in order to determine its unit weight. A small portion of each sample was then subjected to testing in order to determine its moisture content. This was used in order to determine the dry density of the soil in its natural condition. The results of this testing are shown on the Bore Logs.

Maximum Density-Optimum Moisture Determinations: Representative soil types were selected for maximum density determinations. This testing was performed in accordance with the ASTM Standard D1557-91, Test Method A. The results of testing are presented graphically in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

Classification Testing: Soil samples were selected for classification testing. This testing consists of mechanical grain size analyses. This provides information for developing classifications for the soil in accordance with the Unified Soil Classification System which is presented in the preceding appendix. This classification system categorizes the soil into groups having similar engineering characteristics. The results of this testing is very useful in detecting variations in the soils and in selecting samples for further testing.

SOIL MECHANIC'S TESTING

Expansion Testing: One (1) bulk sample was selected for Expansion testing. Expansion testing was performed in accordance with the UBC Standard 18-2. This testing consists of remolding 4-inch diameter by 1-inch thick test specimens to a moisture content and dry density corresponding to approximately 50 percent saturation. The samples are subjected to a surcharge of 144 pounds per square foot and allowed to reach equilibrium. At that point the specimens are inundated with distilled water. The linear expansion is then measured until complete (ASTM D4829).

Direct Shear Testing: One (1) bulk sample was selected for Direct Shear testing. This test measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation design and lateral design. Tests were performed using a recompacted test specimen that was saturated prior to tests. Tests were performed using a strain controlled test apparatus with normal pressures ranging from 800 to 2300 pounds per square foot (ASTM D3080-04).

Consolidation Testing: Two (2) relatively undisturbed samples were selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load. The specimens were saturated at 575 psf or 720 psf load increment (ASTM D2435 & D5333).

Corrosion Series Testing: The soluble sulfate concentrations of the surface soil was determined in accordance with California Test Method Number (CA) 417. The pH and Minimum Resistivity were determined in accordance with CA 643. The soluble chloride concentrations were determined in accordance with CA 422.

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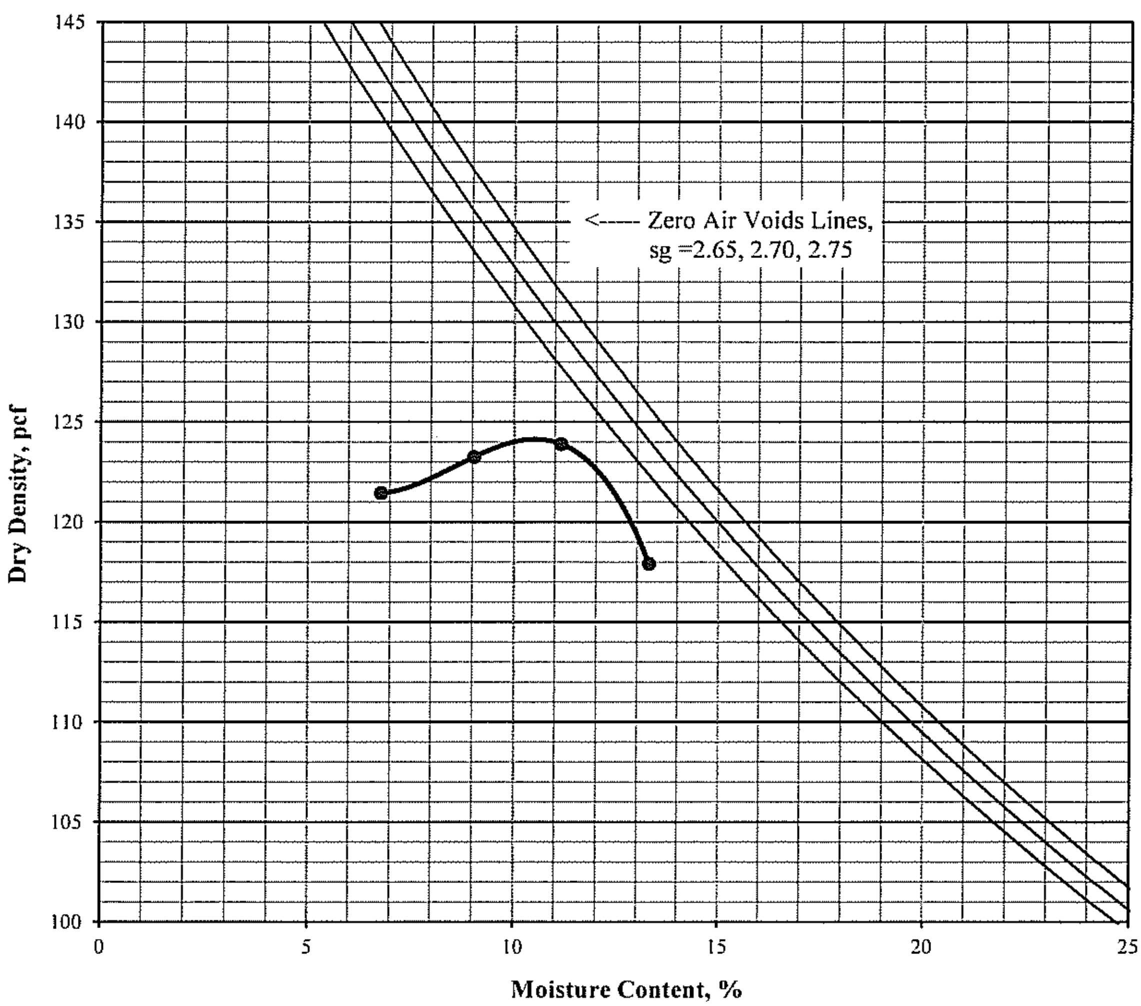
Maximum Density/Optimum Moisture

ASTM D698/D1557

Project Number:	644-15024	February 18, 2016
Project Name:	KPC Promenade	
Lab ID Number:	LN6-16033	ASTM D-1557 A
Sample Location:	BH-1 Bulk 1 @ 0-5'	Rammer Type: Machine
Description:	Dark Brown Silty Sand (SM)	

Maximum Density: **124 pcf**
Optimum Moisture: **10.5%**

Sieve Size	% Retained
3/4"	
3/8"	
#4	0.4



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Expansion Index

ASTM D 4829

Job Number:	644-15024	February 18, 2016
Job Name:	KPC Promenade	
Lab ID Number:	LN6-16033	
Sample ID:	BH-1 Bulk 1 @ 0-5'	
Soil Description:	Dark Brown Silty Sand (SM)	

Wt of Soil + Ring:	577.4
Weight of Ring:	195.0
Wt of Wet Soil:	382.4
Percent Moisture:	9.0%
Sample Height, in	0.95
Wet Density, pcf:	122.0
Dry Denstiy, pcf:	111.9

% Saturation:	48.1

Expansion **Rack # 1**

Date/Time	2/15/2016	10:15 AM
Initial Reading	0.0000	
Final Reading	0.0076	

Expansion Index **8**

(Final - Initial) x 1000

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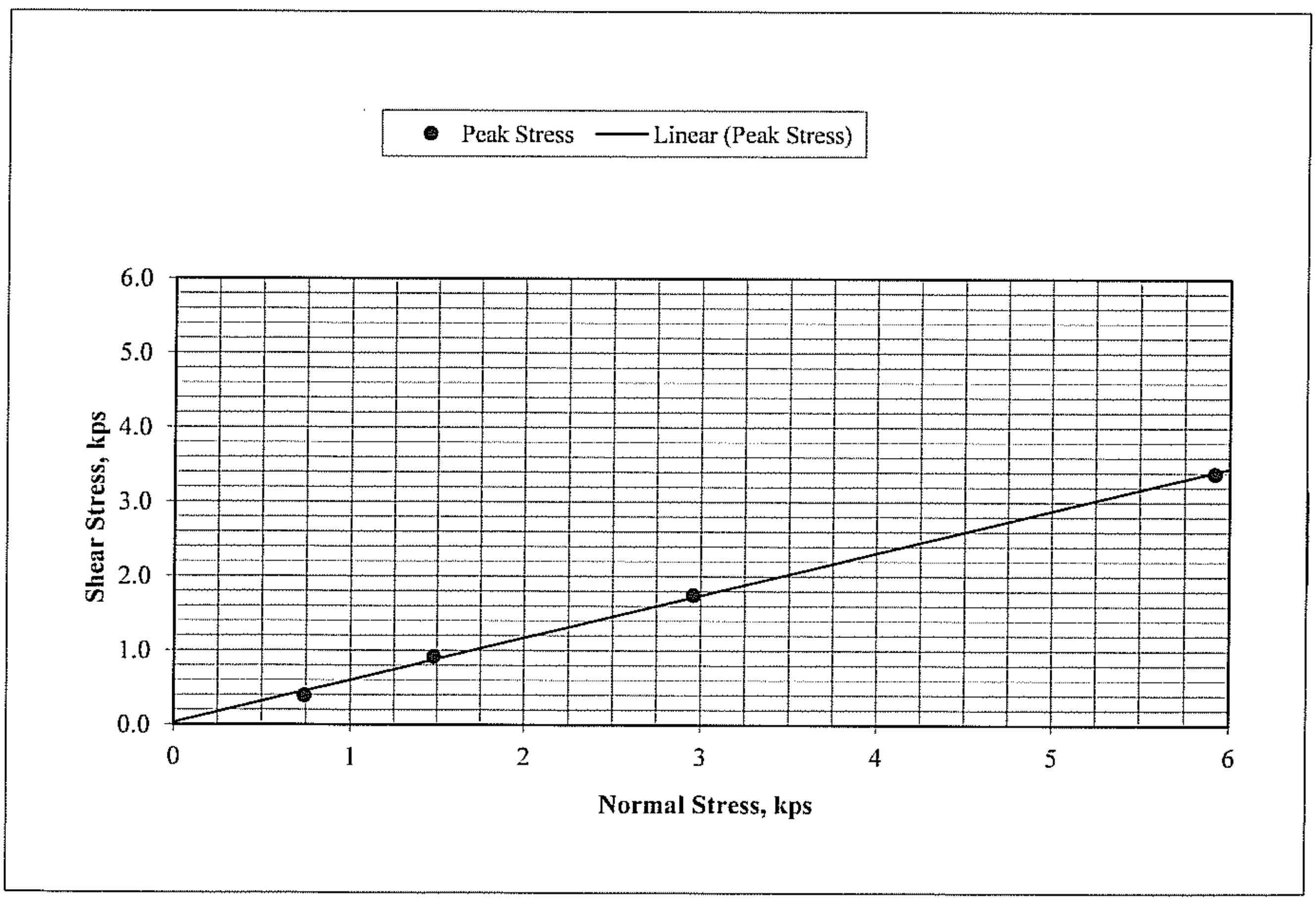
Direct Shear ASTM D 3080-04
(modified for unconsolidated condition)

Job Number: 644-15024
Job Name KPC Promenade
Lab ID No. LN6-16033
Sample ID BH-1 Bulk 1 @ 0-5'
Classification Dark Brown Silty Sand (SM)
Sample Type Remolded @ 90% of Maximum Density

February 18, 2016
Initial Dry Density: 111.5 pcf
Initial Mosture Content: 10.5 %
Peak Friction Angle (Ø): 30°
Cohesion (c): 30 psf

Test Results	1	2	3	4	Average
Moisture Content, %	18.8	18.8	18.8	18.8	18.8
Saturation, %	99.1	99.1	99.1	99.1	99.1
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.393	0.918	1.748	3.387	



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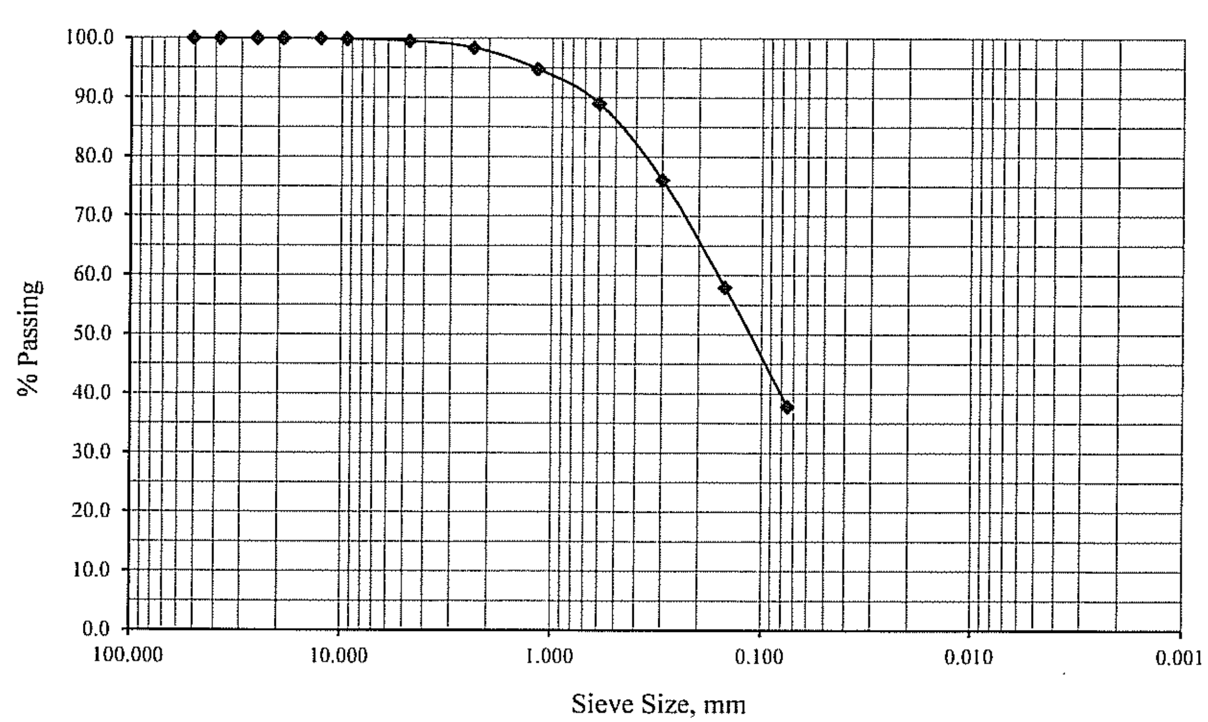
Gradation

ASTM C117 & C136

Project Number: 644-15024 February 18, 2016
Project Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-1 Bulk 1 @ 0-5' Soil Classification: SM

Sieve Size, in	Sieve Size, mm	Percent Passing
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	99.9
#4	4.75	99.6
#8	2.36	98.4
#16	1.18	94.8
#30	0.60	89.0
#50	0.30	76.1
#100	0.15	58.0
#200	0.075	37.9



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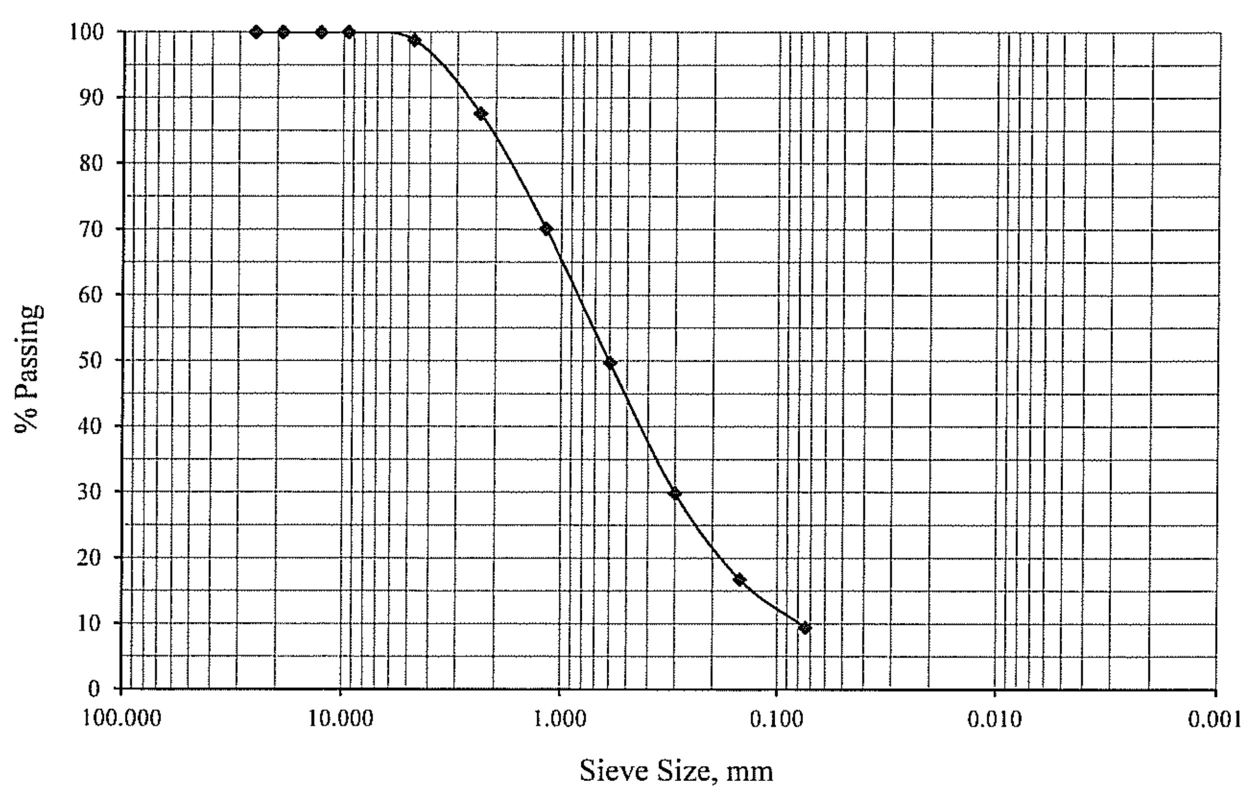
Gradation

ASTM C117 & C136

Project Number:	644-15024	February 18, 2016
Project Name:	KPC Promenade	
Lab ID Number:	LN6-16033	
Sample ID:	BH-1 S-3 @ 10'	Soil Classification: SW-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	98.8
#8	2.36	87.6
#16	1.18	70.1
#30	0.60	49.7
#50	0.30	29.9
#100	0.15	16.8
#200	0.074	9.4



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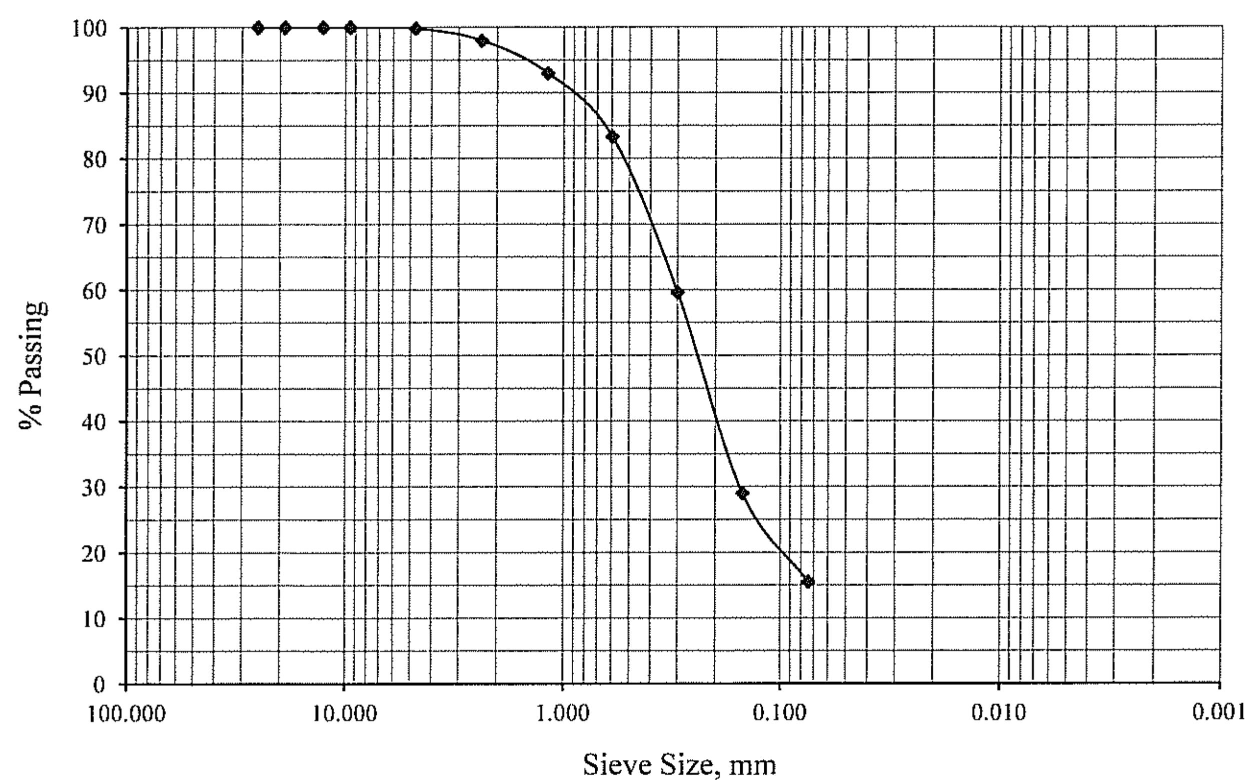
ASTM C117 & C136

Project Number: 644-15024 February 18, 2016
Project Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-1 S-9 @ 40' Soil Classification: SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	99.8
#8	2.36	98.0
#16	1.18	93.0
#30	0.60	83.4
#50	0.30	59.6
#100	0.15	29.0
#200	0.074	15.5



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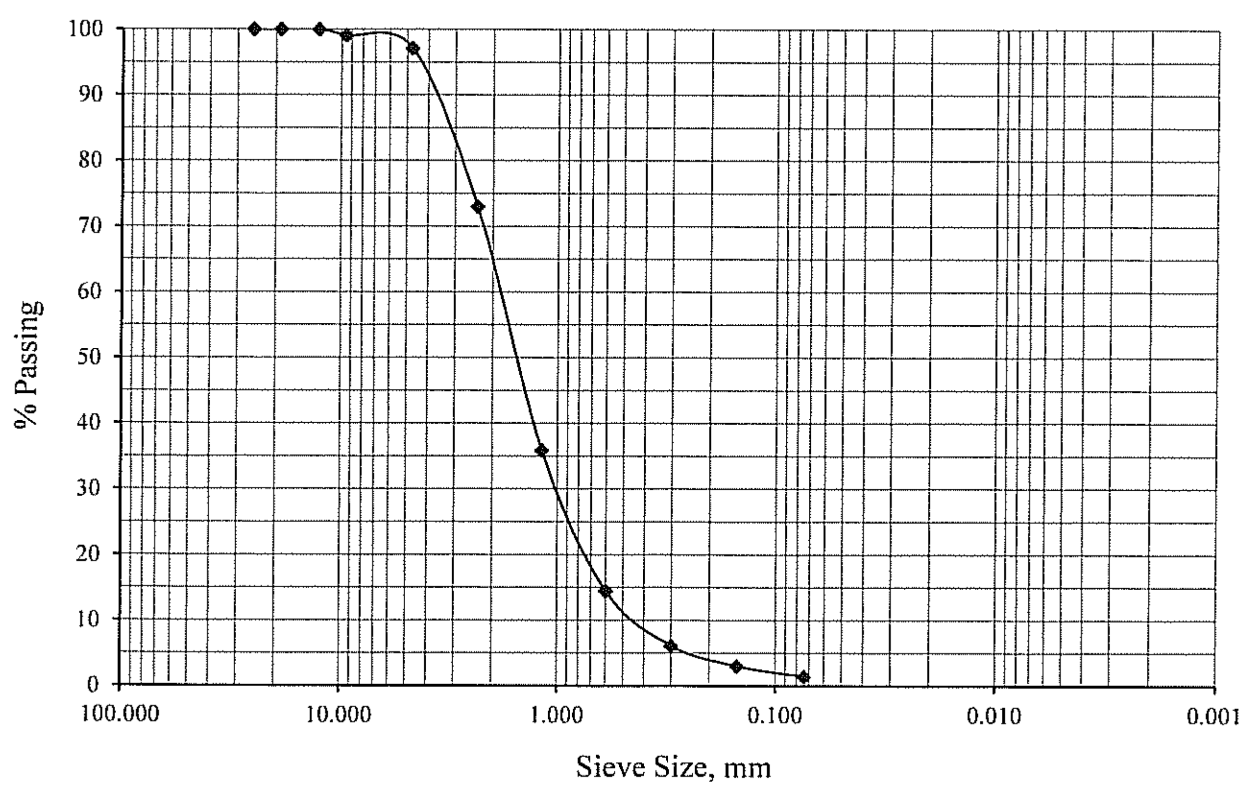
Gradation

ASTM C117 & C136

Project Number: 644-15024 February 18, 2016
Project Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-2 R-2 @ 10' Soil Classification: SP

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	99.0
#4	4.75	97.1
#8	2.36	73.0
#16	1.18	35.9
#30	0.60	14.5
#50	0.30	6.1
#100	0.15	3.0
#200	0.074	1.4



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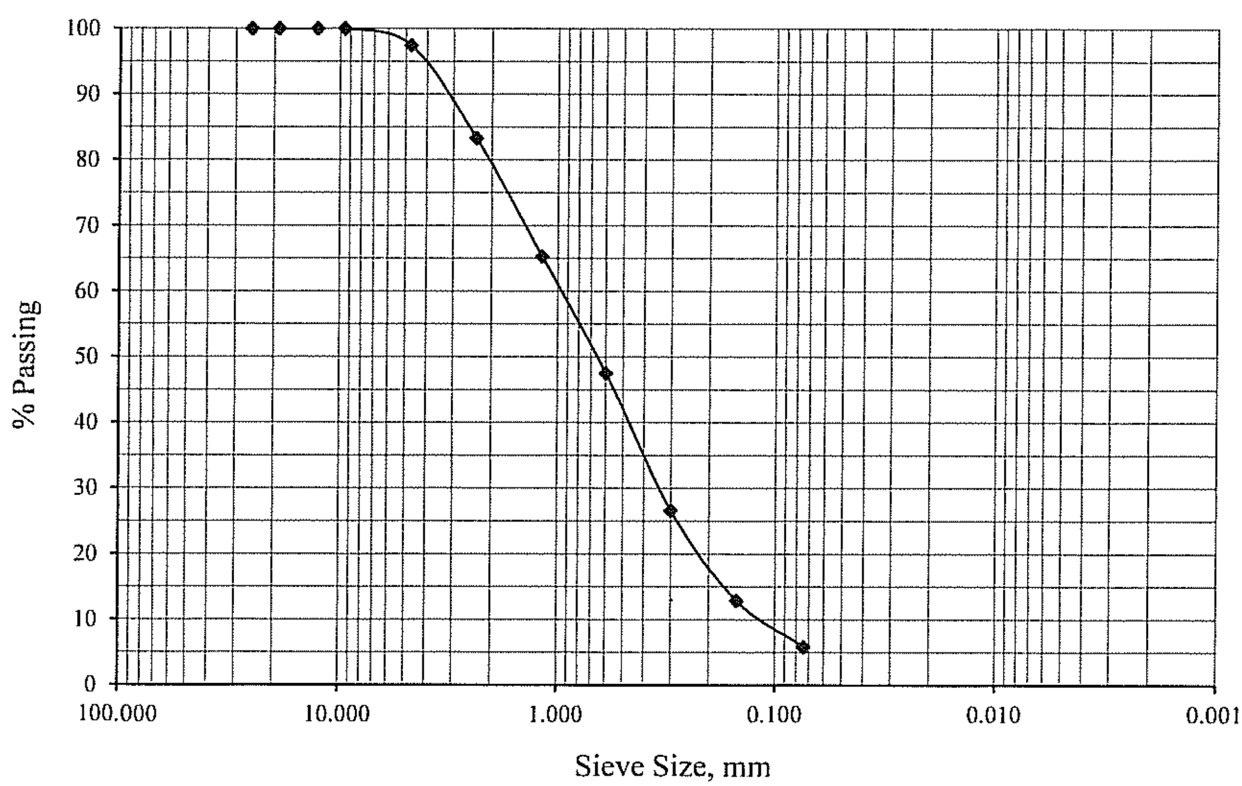
Gradation

ASTM C117 & C136

Project Number: 644-15024 February 18, 2016
Project Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-3 R-1 @ 5' Soil Classification: SP-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	97.5
#8	2.36	83.3
#16	1.18	65.3
#30	0.60	47.6
#50	0.30	26.6
#100	0.15	12.9
#200	0.074	5.8



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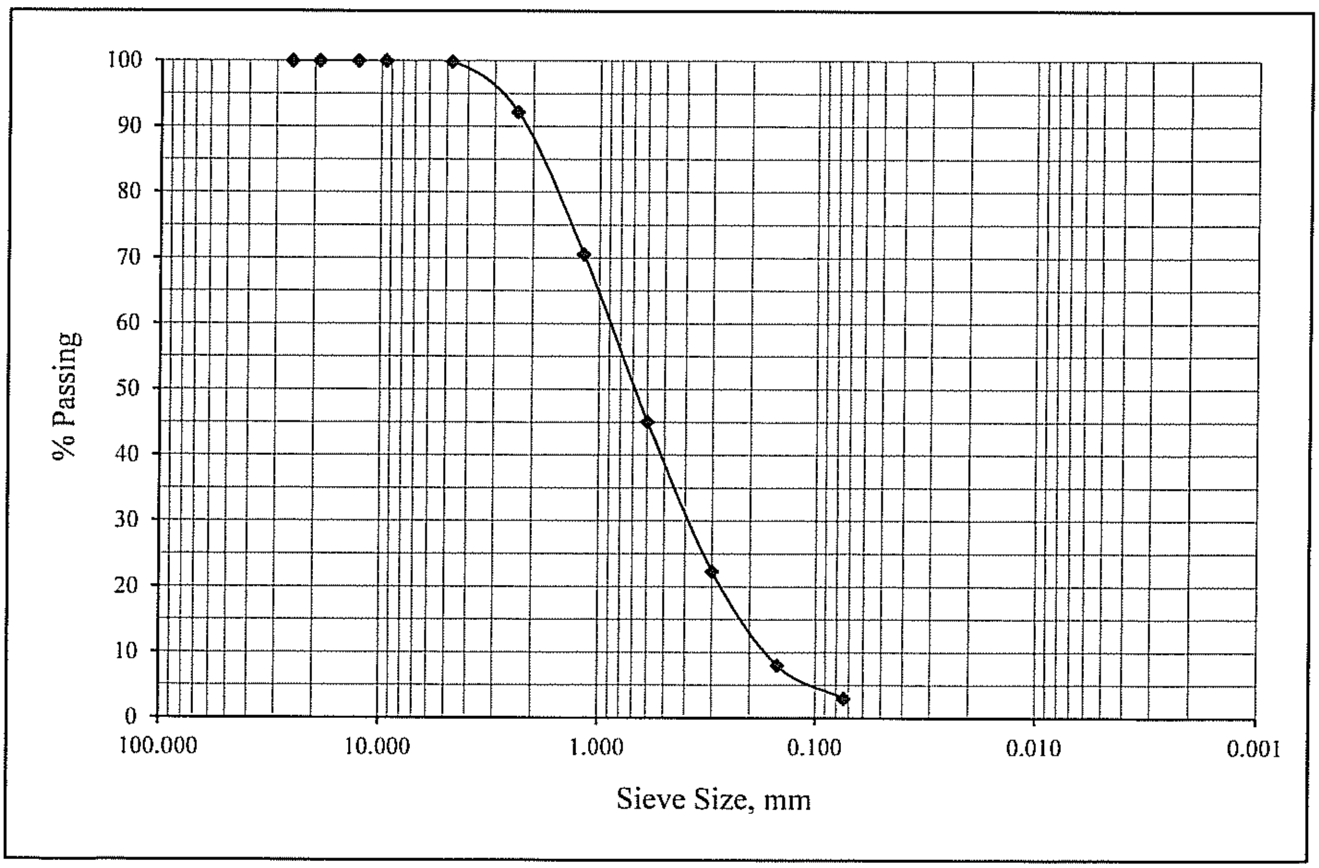
ASTM C117 & C136

Project Number: 644-15024 February 18, 2016
Project Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-4 R-1 @ 5' Soil Classification: SP

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	99.9
#8	2.36	92.2
#16	1.18	70.6
#30	0.60	45.1
#50	0.30	22.4
#100	0.15	8.1
#200	0.074	3.0



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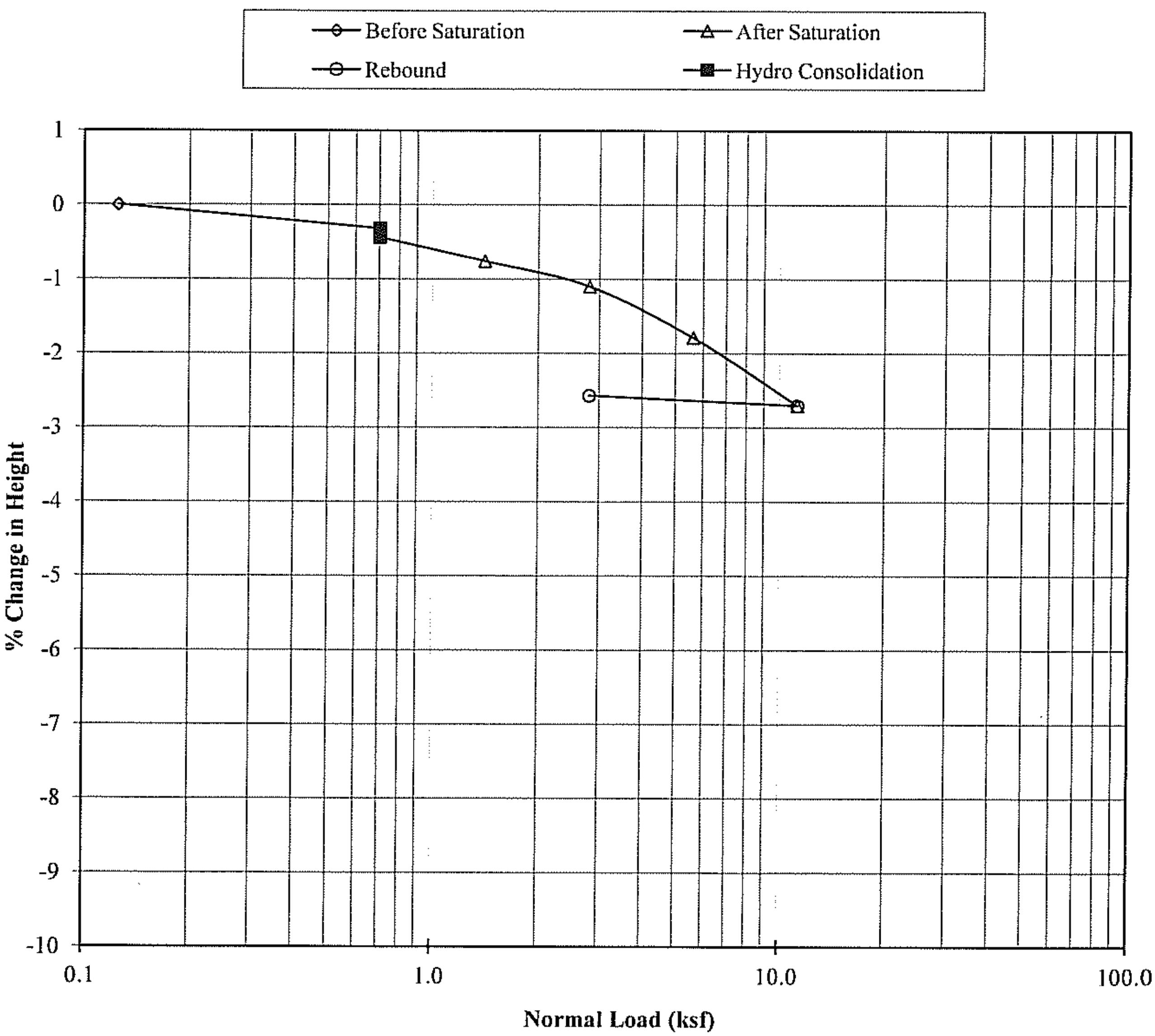
One Dimensional Consolidation

ASTM D2435 & D5333

Job Number:	644-15024		February 18, 2016
Job Name:	KPC Promenade		
Lab ID Number:	LN6-16033	Initial Dry Density, pcf:	101.5
Sample ID:	BH-2 R-2 @ 10'	Initial Moisture, %:	1.4
Soil Description:	Light Brown Sand (SP)	Initial Void Ratio:	0.643
		Specific Gravity:	2.67

Hydrocollapse: 0.1% @ 0.702 ksf

% Change in Height vs Normal Presssure Diagram



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One Dimensional Consolidation

ASTM D2435 & D5333

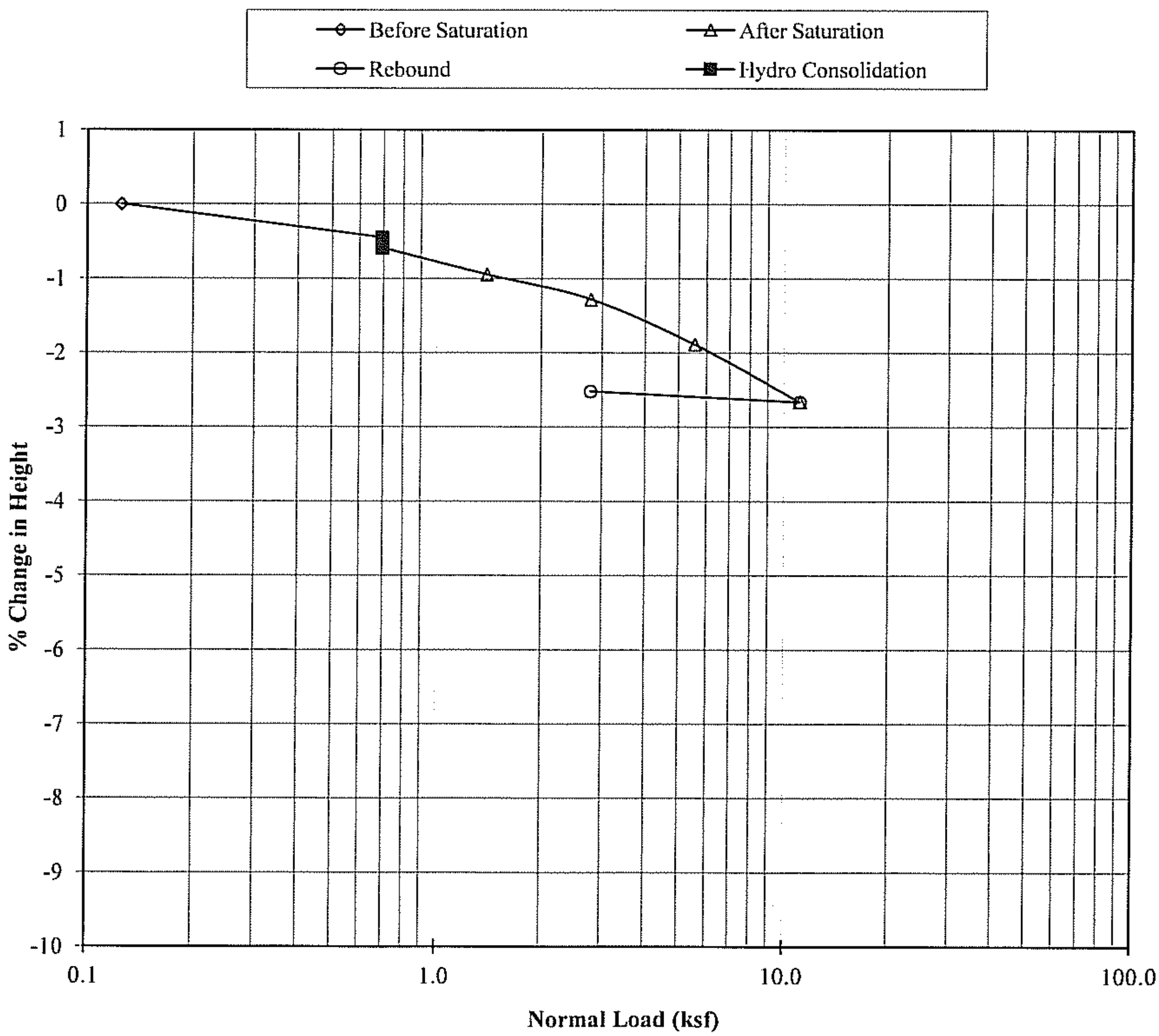
Job Number: 644-15024
Job Name: KPC Promenade
Lab ID Number: LN6-16033
Sample ID: BH-4 R-1 @ 5'
Soil Description: Light Brown Sand (SP)

February 18, 2016

Initial Dry Density, pcf:	108.2
Initial Moisture, %:	1.6
Initial Void Ratio:	0.540
Specific Gravity:	2.67

Hydrocollapse: 0.1% @ 0.694 ksf

% Change in Height vs Normal Presssure Diagram



Sladden Engineering

6782 Stanton Ave., Suite A, Buena Park, CA 90621 (714) 523-0952 Fax (714) 523-1369
45090 Golf Center Pkwy., Suite F, Indio, CA 92201 (760) 863-0713 Fax (760) 863-0847
450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

Date: February 18, 2016

Account No.: 644-15024

Customer: Latham Management Consulting, Inc. c/o Rosenthal & Excell

Location: KPC Promenade, NWC Ramona Expressway & Main Street, San Jacinto

Analytical Report

Corrosion Series

	pH per CA 643	Soluble Sulfates per CA 417 ppm	Soluble Chloride per CA 422 ppm	Min. Resistivity per CA 643 ohm-cm
BH-1 @ 0-5'	8.2	20	30	3400

APPENDIX C

2013 SEISMIC DESIGN MAP AND REPORT
VS30 GRADIENT MAP
PSH DEAGGREGATION OUTPUT

USGS Design Maps Summary Report

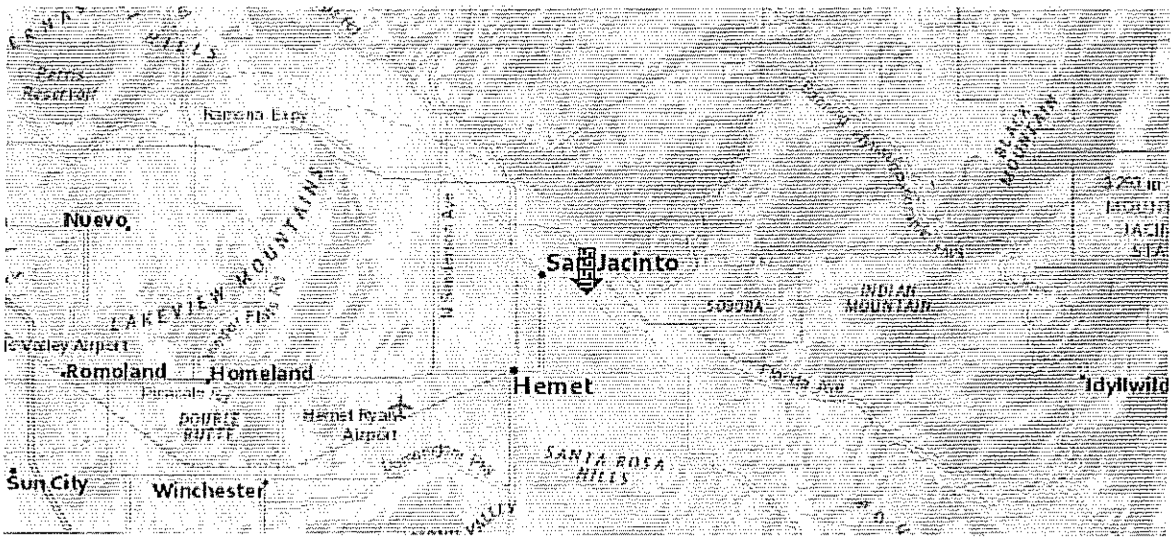
User–Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.7856°N, 116.93887°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

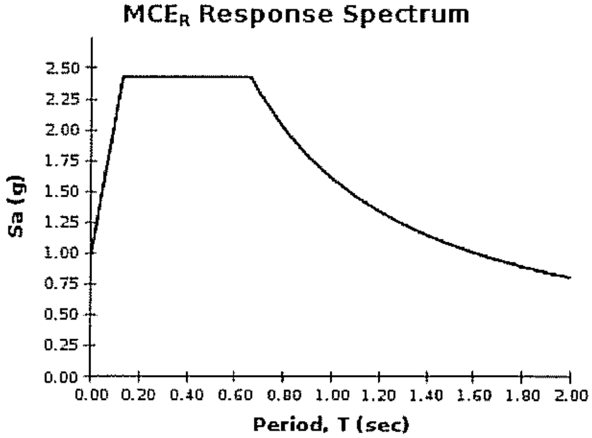


USGS–Provided Output

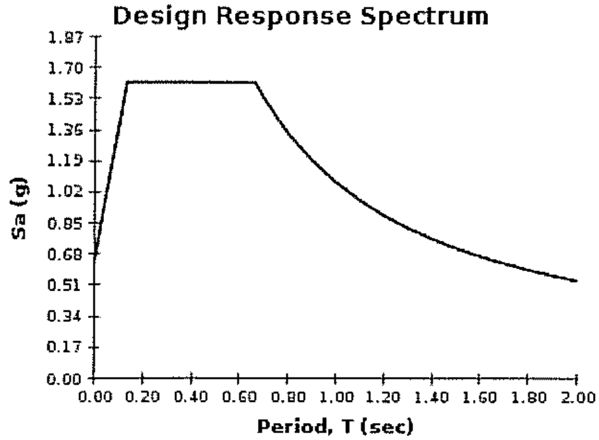
S_S =	2.429 g	S_{MS} =	2.429 g	S_{DS} =	1.620 g
S_1 =	1.071 g	S_{M1} =	1.606 g	S_{D1} =	1.071 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.7856°N, 116.93887°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.429$ g

From Figure 22-2 [2] $S_1 = 1.071$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.429 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 1.071 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.429 = 2.429$ g

---

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 1.071 = 1.606$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.429 = 1.620$ g

---

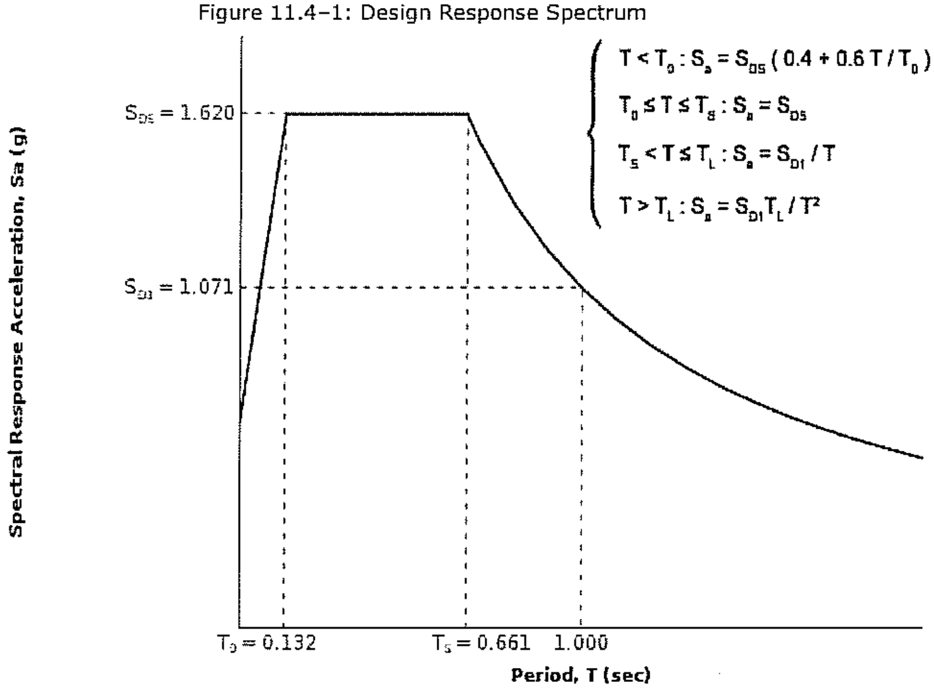
Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.606 = 1.071$ g

---

Section 11.4.5 — Design Response Spectrum

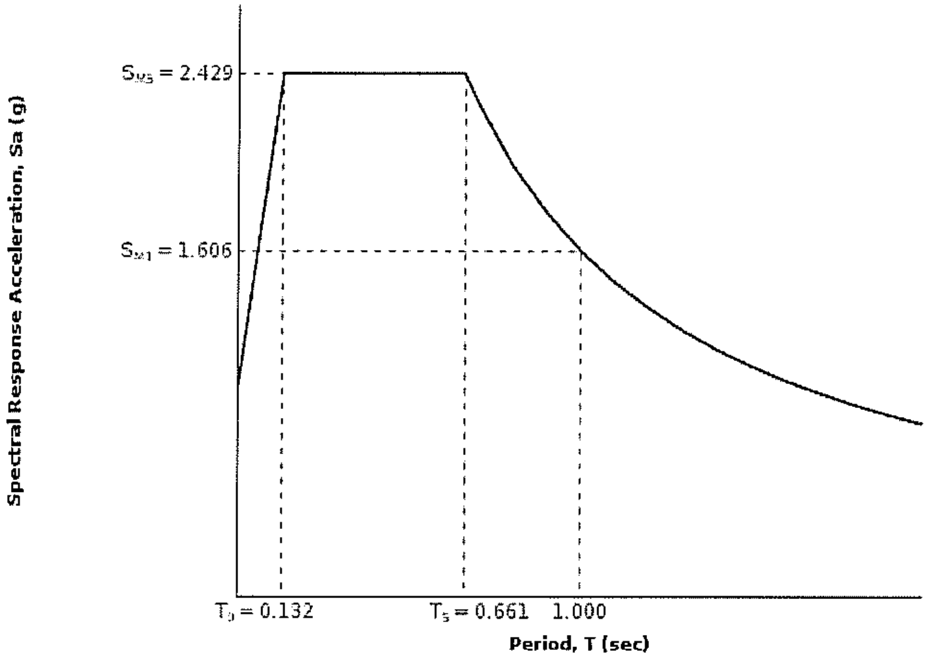
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.935

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.935 = 0.935$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.935 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.955

From Figure 22-18 [6] C_{R1} = 0.925

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.620 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 1.071 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

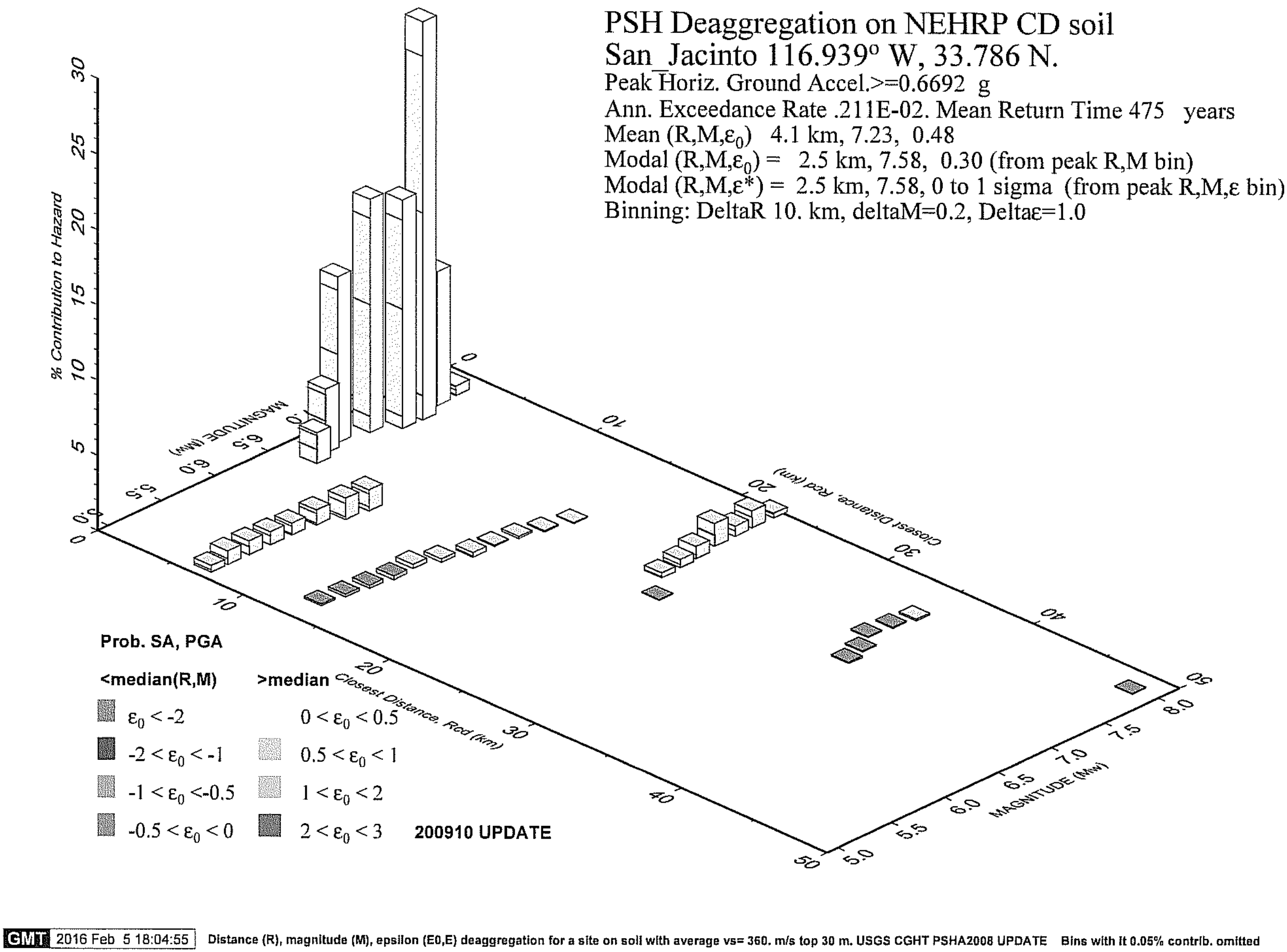
Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

6 Feb 5 18:02:29

-116.970833	33.795834	259
-116.962502	33.795834	256
-116.954170	33.795834	261
-116.945831	33.795834	276
-116.937500	33.795834	359
-116.929169	33.795834	493
-116.920830	33.795834	568
-116.912498	33.795834	517
-116.904167	33.795834	558
-116.970833	33.787498	249
-116.962502	33.787498	262
-116.954170	33.787498	261
-116.945831	33.787498	261
-116.937500	33.787498	270
-116.929169	33.787498	390
-116.920830	33.787498	521
-116.912498	33.787498	518
-116.904167	33.787498	524
-116.970833	33.779167	285
-116.962502	33.779167	274
-116.954170	33.779167	282
-116.945831	33.779167	272
-116.937500	33.779167	269
-116.929169	33.779167	264
-116.920830	33.779167	384
-116.912498	33.779167	468
-116.904167	33.779167	453
-116.970833	33.770832	282
-116.962502	33.770832	285
-116.954170	33.770832	282
-116.945831	33.770832	275
-116.937500	33.770832	284
-116.929169	33.770832	261
-116.920830	33.770832	267
-116.912498	33.770832	382
-116.904167	33.770832	403



PSH Deaggregation on NEHRP CD soil
San_Jacinto 116.939° W, 33.786 N.
Peak Horiz. Ground Accel.>=0.6692 g
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
Mean (R,M,ε0) 4.1 km, 7.23, 0.48
Modal (R,M,ε0) = 2.5 km, 7.58, 0.30 (from peak R,M bin)
Modal (R,M,ε*) = 2.5 km, 7.58, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
MAGNITUDE (Mw)
Closest Distance, Rcd (km)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Feb 5 18:04:55
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 360. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: San_Jacinto long: 116.939 W., lat: 33.786 N.
Vs30(m/s)= 360.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.6692 g. Weight * Computed_Rate_Ex 0.211E-02
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.02236
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.6	5.05	0.436	0.295	0.141	0.000	0.000	0.000	0.000
6.6	5.20	0.914	0.511	0.402	0.000	0.000	0.000	0.000
13.0	5.20	0.117	0.117	0.000	0.000	0.000	0.000	0.000
6.6	5.40	0.937	0.401	0.537	0.000	0.000	0.000	0.000
13.2	5.40	0.167	0.167	0.000	0.000	0.000	0.000	0.000
6.6	5.60	0.894	0.287	0.565	0.042	0.000	0.000	0.000
13.4	5.60	0.213	0.213	0.000	0.000	0.000	0.000	0.000
6.6	5.80	0.797	0.207	0.514	0.075	0.000	0.000	0.000
13.6	5.80	0.246	0.238	0.008	0.000	0.000	0.000	0.000
6.7	6.01	0.993	0.242	0.638	0.113	0.000	0.000	0.000
13.4	6.00	0.295	0.261	0.034	0.000	0.000	0.000	0.000
7.3	6.20	1.429	0.358	0.894	0.177	0.000	0.000	0.000
13.9	6.20	0.234	0.190	0.045	0.000	0.000	0.000	0.000
7.5	6.40	1.390	0.269	0.876	0.245	0.000	0.000	0.000
14.7	6.40	0.310	0.223	0.088	0.000	0.000	0.000	0.000
2.1	6.65	2.067	0.216	0.930	0.901	0.021	0.000	0.000
14.6	6.60	0.090	0.067	0.023	0.000	0.000	0.000	0.000
1.6	6.80	4.262	0.381	1.743	2.081	0.057	0.000	0.000
14.8	6.80	0.118	0.079	0.039	0.000	0.000		

							0.000	0.000
24.5	6.80	0.056	0.056	0.000	0.000	0.000	0.000	0.000
1.3	6.96	11.039	0.918	4.248	5.523	0.350	0.000	0.000
15.3	6.98	0.089	0.053	0.036	0.000	0.000	0.000	0.000
22.9	7.04	0.332	0.262	0.070	0.000	0.000	0.000	0.000
35.9	7.01	0.109	0.109	0.000	0.000	0.000	0.000	0.000
1.6	7.21	15.485	1.244	5.778	7.402	1.062	0.000	0.000
15.7	7.20	0.071	0.034	0.036	0.002	0.000	0.000	0.000
22.7	7.23	0.549	0.347	0.202	0.000	0.000	0.000	0.000
35.5	7.19	0.092	0.092	0.000	0.000	0.000	0.000	0.000
2.4	7.41	15.161	1.315	5.949	7.098	0.799	0.000	0.000
22.6	7.40	0.808	0.561	0.247	0.000	0.000	0.000	0.000
34.6	7.37	0.079	0.075	0.004	0.000	0.000	0.000	0.000
2.5	7.58	26.367	2.287	10.312	12.146	1.622	0.000	0.000
22.3	7.61	1.514	0.786	0.728	0.000	0.000	0.000	0.000
34.8	7.57	0.087	0.073	0.014	0.000	0.000	0.000	0.000
2.0	7.80	8.894	0.715	3.285	4.295	0.599	0.000	0.000
22.3	7.80	0.691	0.320	0.371	0.000	0.000	0.000	0.000
35.1	7.76	0.113	0.089	0.024	0.000	0.000	0.000	0.000
1.9	7.99	0.647	0.048	0.227	0.322	0.050	0.000	0.000
22.3	7.97	1.126	0.434	0.691	0.002	0.000	0.000	0.000
48.2	7.98	0.067	0.060	0.008	0.000	0.000	0.000	0.000
22.3	8.17	0.290	0.076	0.196	0.018	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 4.1 km; M= 7.23; eps0= 0.48. Mean calculated for all sources.
Modal src-site R= 2.5 km; M= 7.58; eps0= 0.30 from peak

```
(R,M) bin
 MODE R*=   2.5km; M*= 7.58; EPS.INTERVAL: 0 to 1 sigma  %
CONTRIB.= 12.146

Principal sources (faults, subduction, random seismicity having >
3% contribution)
Source Category:                     % contr.  R(km)    M   epsilon0
(mean values).
California A-faults                    88.99     3.4   7.38    0.38
CA Compr. crustal gridded              10.44     8.2   5.92    1.29
Individual fault hazard details if its contribution to mean
hazard > 2%:
Fault ID                             % contr.   Rcd(km)  M   epsilon0
Site-to-src azimuth(d)
San Jacinto;SJV aPriori                15.38     1.1   6.95    0.29
58.9
San Jacinto;SBV+SJV aPriori             9.00     1.1   7.31    0.16
58.9
San Jacinto;A+C aPriori                15.79     3.4   7.49    0.41
-131.4
San Jacinto;SBV+SJV+A+C aPriori         6.83     1.9   7.76    0.19
-130.9
San Jacinto;SJV MoBal                   3.63     1.1   6.95    0.29
58.9
San Jacinto;SBV+SJV MoBal               3.01     1.1   7.31    0.16
58.9
San Jacinto;SJV+A MoBal                 2.58     1.9   7.46    0.24
-130.9
San Jacinto;A+C MoBal                   5.86     3.4   7.49    0.41
-131.4
San Jacinto;SBV+SJV+A MoBal             2.64     1.9   7.62    0.21
-130.9
San Jacinto;SJV+A+C MoBal               2.65     1.9   7.63    0.21
-130.9
San Jacinto;SBV+SJV+A+C MoBal           2.70     1.9   7.76    0.19
-130.9
San Jacinto (SB to C) Unsegmente        8.36     2.3   7.30    0.34
-131.5
#*********End of deaggregation corresponding to Mean Hazard w/all
GMPEs  *********#

PSHA Deaggregation. %contributions. site: San_Jacinto long:
116.939 W., lat: 33.786 N.
 Vs30(m/s)= 360.0 (some WUS atten. models use Site Class not
Vs30).
NSHMP 2007-08  See USGS OFR 2008-1128. dM=0.2 below
Return period: 475   yrs. Exceedance PGA =0.6692   g. Weight *
Computed_Rate_Ex 0.836E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.02558
#This deaggregation corresponds to Boore-Atkinson 2008
DIST(KM) MAG(MW) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2
<EPS<-1 EPS<-2
```

6.3	5.05	0.030	0.030	0.000	0.000	0.000	0.000	0.000
6.4	5.20	0.079	0.079	0.000	0.000	0.000	0.000	0.000
6.4	5.40	0.101	0.099	0.002	0.000	0.000	0.000	0.000
6.5	5.60	0.118	0.103	0.014	0.000	0.000	0.000	0.000
6.5	5.80	0.128	0.103	0.025	0.000	0.000	0.000	0.000
6.3	6.02	0.211	0.137	0.075	0.000	0.000	0.000	0.000
13.4	6.01	0.035	0.035	0.000	0.000	0.000	0.000	0.000
6.7	6.20	0.317	0.190	0.126	0.000	0.000	0.000	0.000
14.2	6.20	0.038	0.038	0.000	0.000	0.000	0.000	0.000
7.0	6.40	0.310	0.178	0.132	0.000	0.000	0.000	0.000
15.2	6.40	0.064	0.064	0.000	0.000	0.000	0.000	0.000
1.8	6.66	0.788	0.098	0.352	0.338	0.000	0.000	0.000
15.4	6.61	0.034	0.033	0.001	0.000	0.000	0.000	0.000
24.1	6.60	0.029	0.029	0.000	0.000	0.000	0.000	0.000
1.5	6.80	1.861	0.156	0.756	0.949	0.000	0.000	0.000
15.7	6.80	0.051	0.046	0.005	0.000	0.000	0.000	0.000
24.6	6.80	0.047	0.046	0.000	0.000	0.000	0.000	0.000
34.6	6.80	0.041	0.041	0.000	0.000	0.000	0.000	0.000
1.2	6.96	4.560	0.319	1.813	2.361	0.067	0.000	0.000
15.9	6.99	0.043	0.034	0.009	0.000	0.000	0.000	0.000
22.9	7.04	0.258	0.196	0.062	0.000	0.000	0.000	0.000
35.9	7.01	0.108	0.108	0.000	0.000	0.000	0.000	0.000
1.5	7.21	6.077	0.419	2.342	2.978	0.338	0.000	0.000
16.0	7.20	0.039	0.022	0.017	0.001	0.000	0.000	0.000
22.7	7.25	0.490	0.325	0.165	0.000	0.000	0.000	0.000
35.6	7.19	0.086	0.086	0.000	0.000	0.000	0.000	0.000

2.5	7.41	6.774	0.506	2.795	3.298	0.176	0.000	0.000
22.6	7.44	0.591	0.343	0.248	0.000	0.000	0.000	0.000
34.7	7.37	0.069	0.066	0.004	0.000	0.000	0.000	0.000
2.5	7.59	9.905	0.793	4.337	4.462	0.313	0.000	0.000
22.4	7.63	0.783	0.398	0.385	0.000	0.000	0.000	0.000
34.9	7.57	0.073	0.060	0.014	0.000	0.000	0.000	0.000
48.4	7.54	0.021	0.021	0.000	0.000	0.000	0.000	0.000
1.9	7.79	3.665	0.244	1.374	1.842	0.205	0.000	0.000
22.3	7.80	0.416	0.164	0.252	0.000	0.000	0.000	0.000
35.1	7.76	0.091	0.067	0.024	0.000	0.000	0.000	0.000
48.1	7.78	0.021	0.021	0.000	0.000	0.000	0.000	0.000
1.9	7.99	0.262	0.017	0.097	0.132	0.016	0.000	0.000
22.3	7.98	0.679	0.225	0.447	0.007	0.000	0.000	0.000
48.2	7.98	0.055	0.048	0.008	0.000	0.000	0.000	0.000
22.3	8.19	0.119	0.032	0.078	0.009	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 39.6
Mean src-site R= 4.7 km; M= 7.33; eps0= 0.53. Mean calculated for all sources.
Modal src-site R= 2.5 km; M= 7.59; eps0= 0.41 from peak (R,M) bin
MODE R*= 2.4km; M*= 7.59; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 4.462

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category: % contr. R(km) M epsilon0 (mean values).
California A-faults 37.51 4.3 7.38 0.47
Individual fault hazard details if its contribution to mean hazard > 2%:
Fault ID % contr. Rcd(km) M epsilon0 Site-to-src azimuth(d)
San Jacinto;SJV aPriori 6.52 1.1 6.95 0.30 58.9

San Jacinto;SBV+SJV aPriori 3.75 1.1 7.31 0.19
58.9
San Jacinto;A+C aPriori 6.11 3.4 7.49 0.51
-131.4
San Jacinto;SBV+SJV+A+C aPriori 2.76 1.9 7.76 0.26
-130.9
San Jacinto;SJV MoBal 1.54 1.1 6.95 0.30
58.9
San Jacinto;SBV+SJV MoBal 1.25 1.1 7.31 0.19
58.9
San Jacinto;SJV+A MoBal 1.04 1.9 7.46 0.30
-130.9
San Jacinto;A+C MoBal 2.27 3.4 7.49 0.51
-131.4
San Jacinto;SBV+SJV+A MoBal 1.07 1.9 7.62 0.27
-130.9
San Jacinto;SJV+A+C MoBal 1.07 1.9 7.63 0.27
-130.9
San Jacinto;SBV+SJV+A+C MoBal 1.09 1.9 7.76 0.26
-130.9
San Jacinto (SB to C) Unsegmente 3.40 2.4 7.30 0.40
-131.5
#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: San_Jacinto long: 116.939 W., lat: 33.786 N.
Vs30(m/s)= 360.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.6692 g. Weight * Computed_Rate_Ex 0.197E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.5	5.05	0.067	0.067	0.000	0.000	0.000	0.000	0.000
6.6	5.20	0.172	0.164	0.008	0.000	0.000	0.000	0.000
6.6	5.40	0.223	0.175	0.048	0.000	0.000	0.000	0.000
12.7	5.42	0.017	0.017	0.000	0.000	0.000	0.000	0.000
6.6	5.60	0.228	0.159	0.069	0.000	0.000	0.000	0.000
13.0	5.60	0.033	0.033	0.000	0.000	0.000	0.000	0.000
6.6	5.80	0.193	0.130	0.064	0.000	0.000	0.000	0.000
13.2	5.80	0.040	0.040	0.000	0.000	0.000	0.000	0.000

6.7	6.01	0.214	0.154	0.060	0.000	0.000	0.000	0.000
13.1	6.01	0.051	0.051	0.000	0.000	0.000	0.000	0.000
7.3	6.20	0.311	0.219	0.092	0.000	0.000	0.000	0.000
13.7	6.20	0.041	0.041	0.000	0.000	0.000	0.000	0.000
7.4	6.40	0.328	0.200	0.128	0.000	0.000	0.000	0.000
14.3	6.40	0.056	0.056	0.000	0.000	0.000	0.000	0.000
3.2	6.63	0.240	0.096	0.143	0.001	0.000	0.000	0.000
14.2	6.60	0.019	0.019	0.001	0.000	0.000	0.000	0.000
2.2	6.79	0.361	0.137	0.224	0.001	0.000	0.000	0.000
14.2	6.80	0.020	0.019	0.001	0.000	0.000	0.000	0.000
1.3	6.96	0.882	0.294	0.542	0.046	0.000	0.000	0.000
14.9	6.96	0.011	0.010	0.001	0.000	0.000	0.000	0.000
23.8	7.03	0.005	0.005	0.000	0.000	0.000	0.000	0.000
1.6	7.21	1.385	0.373	0.768	0.243	0.000	0.000	0.000
24.0	7.21	0.007	0.007	0.000	0.000	0.000	0.000	0.000
2.4	7.40	1.237	0.368	0.694	0.175	0.000	0.000	0.000
23.3	7.40	0.013	0.013	0.000	0.000	0.000	0.000	0.000
2.5	7.58	2.257	0.672	1.267	0.318	0.000	0.000	0.000
22.7	7.61	0.014	0.014	0.000	0.000	0.000	0.000	0.000
2.0	7.79	0.824	0.225	0.458	0.141	0.000	0.000	0.000
22.2	7.81	0.006	0.006	0.000	0.000	0.000	0.000	0.000
1.9	7.99	0.054	0.014	0.030	0.010	0.000	0.000	0.000
22.2	8.00	0.008	0.008	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 9.3
Mean src-site R= 3.5 km; M= 7.04; eps0= 1.27. Mean calculated for all sources.

```
Modal src-site R=    2.5 km; M= 7.58; eps0=   1.18 from peak
(R,M) bin
 MODE R*=   2.5km; M*= 7.57; EPS.INTERVAL: 0 to 1 sigma  %
CONTRIB.=  1.267

Principal sources (faults, subduction, random seismicity having >
3% contribution)
Source Category:                   % contr.  R(km)    M   epsilon0
(mean values).
California A-faults                  7.06     2.1   7.38    1.21
Individual fault hazard details if its contribution to mean
hazard > 2%:
Fault ID                           % contr.   Rcd(km)  M   epsilon0
Site-to-src azimuth(d)
San Jacinto;SJV aPriori              1.21     1.1   6.96    1.30
58.9
San Jacinto;SBV+SJV aPriori          0.78     1.1   7.31    1.10
58.9
San Jacinto;A+C aPriori              1.36     3.4   7.48    1.24
-131.4
San Jacinto;SBV+SJV+A+C aPriori      0.58     1.9   7.75    1.13
-130.9
San Jacinto;SJV MoBal                0.29     1.1   6.96    1.31
58.9
San Jacinto;SBV+SJV MoBal            0.26     1.1   7.31    1.10
58.9
San Jacinto;SJV+A MoBal              0.23     1.9   7.45    1.13
-130.9
San Jacinto;A+C MoBal                0.50     3.4   7.48    1.24
-131.4
San Jacinto;SBV+SJV+A MoBal          0.23     1.9   7.62    1.13
-130.9
San Jacinto;SJV+A+C MoBal            0.23     1.9   7.63    1.13
-130.9
San Jacinto;SBV+SJV+A+C MoBal        0.23     1.9   7.75    1.13
-130.9
San Jacinto (SB to C) Unsegmente     0.70     2.0   7.32    1.20
-131.5
#*********End of deaggregation corresponding to Campbell-
Bozorgnia 2008  *********#

PSHA Deaggregation. %contributions. site: San_Jacinto long:
116.939 W., lat: 33.786 N.
 Vs30(m/s)= 360.0 (some WUS atten. models use Site Class not
Vs30).
NSHMP 2007-08  See USGS OFR 2008-1128. dM=0.2 below
Return period: 475   yrs. Exceedance PGA =0.6692   g. Weight *
Computed_Rate_Ex 0.108E-02
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.04185
#This deaggregation corresponds to Chiou-Youngs 2008
DIST(KM) MAG(MW) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2
<EPS<-1 EPS<-2
```

6.6	5.05	0.339	0.267	0.072	0.000	0.000	0.000	0.000
12.8	5.05	0.041	0.041	0.000	0.000	0.000	0.000	0.000
6.6	5.20	0.663	0.483	0.179	0.000	0.000	0.000	0.000
13.0	5.20	0.114	0.114	0.000	0.000	0.000	0.000	0.000
6.6	5.40	0.614	0.382	0.232	0.000	0.000	0.000	0.000
13.3	5.40	0.149	0.149	0.000	0.000	0.000	0.000	0.000
6.6	5.60	0.548	0.282	0.266	0.000	0.000	0.000	0.000
13.5	5.60	0.174	0.174	0.000	0.000	0.000	0.000	0.000
6.6	5.80	0.475	0.201	0.274	0.000	0.000	0.000	0.000
13.6	5.80	0.189	0.189	0.000	0.000	0.000	0.000	0.000
6.9	6.01	0.567	0.234	0.333	0.000	0.000	0.000	0.000
13.5	6.00	0.210	0.202	0.008	0.000	0.000	0.000	0.000
7.6	6.20	0.802	0.350	0.452	0.000	0.000	0.000	0.000
13.9	6.20	0.155	0.144	0.011	0.000	0.000	0.000	0.000
22.1	6.21	0.027	0.027	0.000	0.000	0.000	0.000	0.000
7.7	6.40	0.752	0.265	0.484	0.003	0.000	0.000	0.000
14.6	6.39	0.190	0.166	0.023	0.000	0.000	0.000	0.000
2.1	6.65	1.040	0.107	0.436	0.477	0.019	0.000	0.000
14.1	6.60	0.036	0.033	0.003	0.000	0.000	0.000	0.000
1.6	6.79	2.040	0.152	0.773	1.060	0.055	0.000	0.000
14.2	6.80	0.047	0.039	0.008	0.000	0.000	0.000	0.000
1.3	6.95	5.597	0.328	1.900	3.088	0.282	0.000	0.000
14.6	6.98	0.035	0.027	0.008	0.000	0.000	0.000	0.000
22.6	7.05	0.068	0.063	0.005	0.000	0.000	0.000	0.000
1.6	7.21	7.096	0.393	2.311	3.694	0.698	0.000	0.000
14.9	7.20	0.029	0.016	0.011	0.001	0.000	0.000	0.000

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   22.7    7.22    0.131    0.104    0.028    0.000    0.000
0.000    0.000
    2.4    7.39    8.078    0.501    2.817    4.112    0.649
0.000    0.000
   22.5    7.39    0.291    0.184    0.106    0.000    0.000
0.000    0.000
    2.6    7.58   13.785    0.805    4.608    7.113    1.260
0.000    0.000
   22.3    7.62    0.548    0.340    0.208    0.000    0.000
0.000    0.000
    2.0    7.79    4.824    0.263    1.554    2.565    0.442
0.000    0.000
   22.3    7.81    0.272    0.153    0.119    0.000    0.000
0.000    0.000
    1.9    7.99    0.332    0.017    0.100    0.181    0.034
0.000    0.000
   22.3    7.97    0.492    0.209    0.283    0.000    0.000
0.000    0.000
   22.3    8.18    0.119    0.035    0.079    0.005    0.000
0.000    0.000

Summary statistics for above PSHA PGA  deaggregation, R=distance,
e=epsilon:
Contribution from this GMPE(%):   51.0
 Mean src-site R=    3.7 km; M= 7.19; eps0=   0.30. Mean
calculated for all sources.
Modal src-site R=    2.6 km; M= 7.58; eps0=   0.09 from peak
(R,M) bin
 MODE R*=   2.6km; M*= 7.58; EPS.INTERVAL: 0 to 1 sigma  %
CONTRIB.=  7.113

Principal sources (faults, subduction, random seismicity having >
3% contribution)
Source Category:                   % contr.  R(km)    M   epsilon0
(mean values).
California A-faults                  44.42     3.0   7.39    0.17
CA Compr. crustal gridded             6.47     8.4   5.85    1.18
Individual fault hazard details if its contribution to mean
hazard > 2%:
Fault ID                           % contr.   Rcd(km)  M   epsilon0
Site-to-src azimuth(d)
San Jacinto;SJV aPriori               7.65      1.1   6.95    0.12
58.9
San Jacinto;SBV+SJV aPriori           4.47      1.1   7.31   -0.03
58.9
San Jacinto;A+C aPriori               8.33      3.4   7.49    0.21
-131.4
San Jacinto;SBV+SJV+A+C aPriori       3.49      1.9   7.76   -0.01
-130.9
San Jacinto;SJV MoBal                 1.81      1.1   6.95    0.13
58.9
San Jacinto;SBV+SJV MoBal             1.49      1.1   7.31   -0.03
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58.9

San Jacinto;SJV+A MoBal	1.31	1.9	7.46	0.04
-130.9				
San Jacinto;A+C MoBal	3.09	3.4	7.49	0.21
-131.4				
San Jacinto;SBV+SJV+A MoBal	1.35	1.9	7.62	0.01
-130.9				
San Jacinto;SJV+A+C MoBal	1.35	1.9	7.63	0.01
-130.9				
San Jacinto;SBV+SJV+A+C MoBal	1.38	1.9	7.76	-0.01
-130.9				
San Jacinto (SB to C) Unsegmente	4.26	2.2	7.30	0.15
-131.5				

#*********End of deaggregation corresponding to Chiou-Youngs 2008 *********#

******************** Southern California **